U.S. Army Corps
of Engineers
Seattle District

Centralia Flood Damage Reduction Project
Chehalis River, Washington
General Reevaluation Report

Appendix A:
Hydrology and Hydraulics

June 2003

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1. INTRODUCTION

This appendix presents information and analyses performed for the re-evaluation study. It presents the conceptual basis for the hydraulic design of the proposed action. The information in this appendix will serve as the basis for subsequent hydraulic modeling in support of the final design, construction plans and specifications to complete the project. The basic flood control objectives of the project are to prevent flooding in the Centralia-Chehalis area from a 1 percent or 100-year flood event and to preserve, as much as possible, existing wetlands and riparian and aquatic habitat along the Chehalis, Newaukum and Skookumchuck Rivers.

2. CHEHALIS RIVER BASIN

2.1 WATERSHED PHYSIOGRAPHY

2.1.1 Drainage Basin

The Chehalis River drainage basin covers approximately 2,114 square miles (Figure 2-1). Above the stream gage at Porter, river mile (RM) 33.3, the drainage area is 1,294 square miles, and above the stream gage at Grand Mound (RM 59.89) the drainage area is 895 square miles. The Chehalis River is about 125 miles long, originating in the Willapa and Doty Hills southeast of the City of Aberdeen and flowing northeast and then northwest before emptying into Grays Harbor at Aberdeen. The basin uplands include the Willapa Hills, the western flank of the Cascade Mountains, and the southern Olympic Mountains.

The Chehalis River originates in the extreme southwestern corner of the basin, and flows east for about 25 miles to its confluence with the Newaukum River at the City of Chehalis. From Chehalis, the river flows north for 8 miles, where it meets the Skookumchuck River at the City of Centralia. The river then turns and flows generally north and west for about 50 miles to its mouth at Grays Harbor on the Washington coast.

The Chehalis River Valley, located in the southern end of the Puget Trough, is characterized by a broad, well-developed floodplain and low terraces surrounded by highly dissected uplands of low to moderate relief that have broad, rounded ridges. There are numerous perennial streams in the valley. The valley bottom in the Centralia-Chehalis area is at an elevation of about 150 feet, and upland elevations average about 300 to 600 feet. Higher elevations in the basin range from about 1,000 feet in the lowland hills, to 2,658 feet at Capital Peak in the south Olympic Range, to 3,800 feet in the foothills of the Cascade Range east of Centralia-Chehalis, and 3,110 feet in the Boistfort Hills along the south basin.

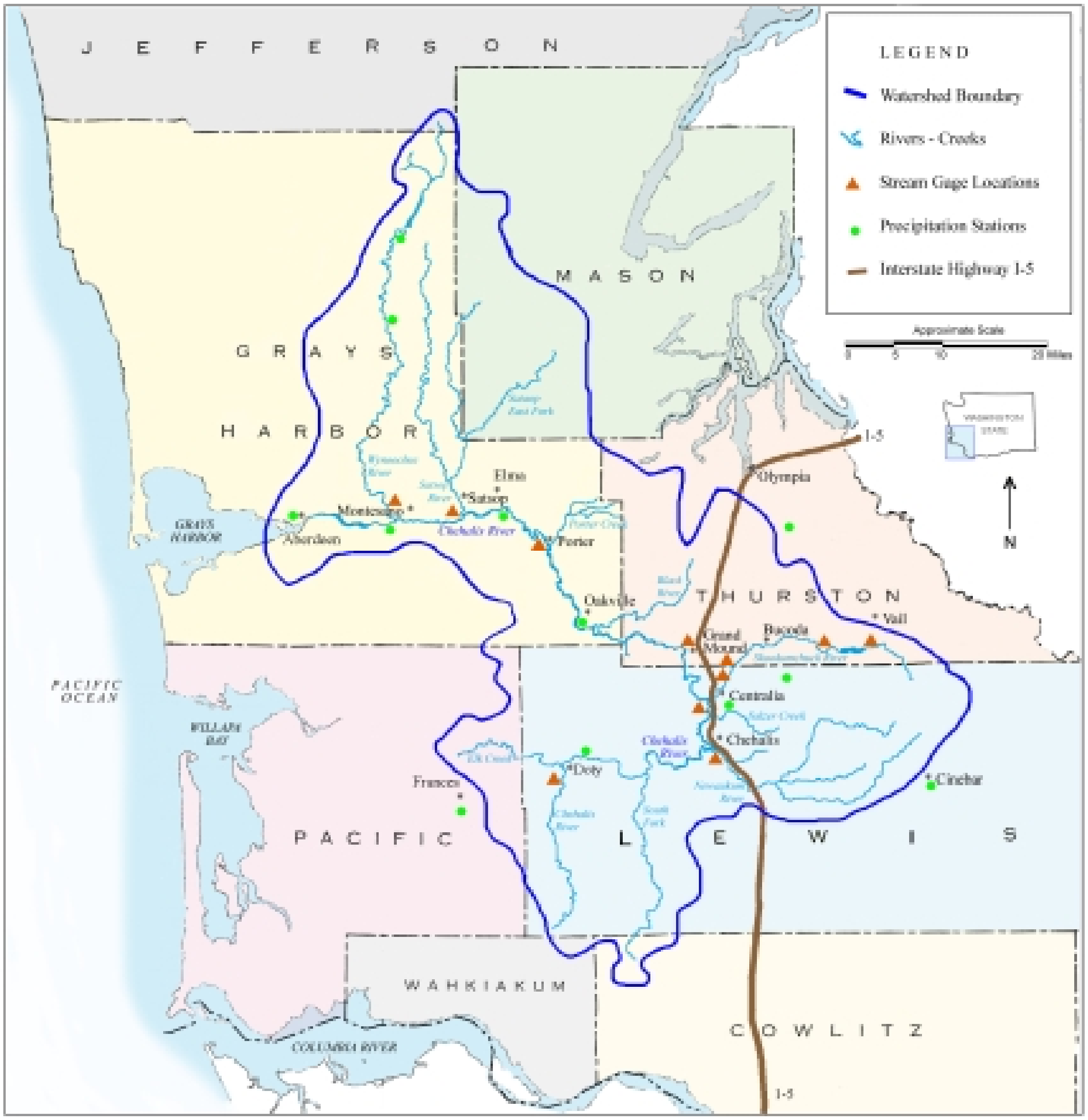


Figure 2-1: Chehalis River Basin Watershed Boundary

2.1.2 Upper Chehalis River Basin

The slope of the upper Chehalis River from its source to the City of Chehalis is steep, falling an average of 16 feet per mile. The slope flattens to about 3 feet per mile in the valley surrounding the cities of Centralia and Chehalis, where the Chehalis River has a meandering channel that occupies a fairly uniform floodplain averaging over 1 mile wide. Most of the valley is inundated during a severe flood such as the January 1990 and the February 1996 floods.

The Upper Chehalis River Basin above Centralia includes four main drainages: the Skookumchuck River, the Newaukum River, the South Fork Chehalis River, and the Chehalis River above Doty. In addition, there are several smaller subdrainages in the Centralia-Chehalis area, including Coffee Creek, China Creek, Salzer Creek, and Dillenbaugh Creek (Figure 2-2). The main drainages between Centralia and the town of Porter include: Lincoln Creek, Scatter Creek, Independence Creek, Black River, Garrad Creek, Rock Creek, Shelton Creek, Cedar Creek, and Porter Creek.

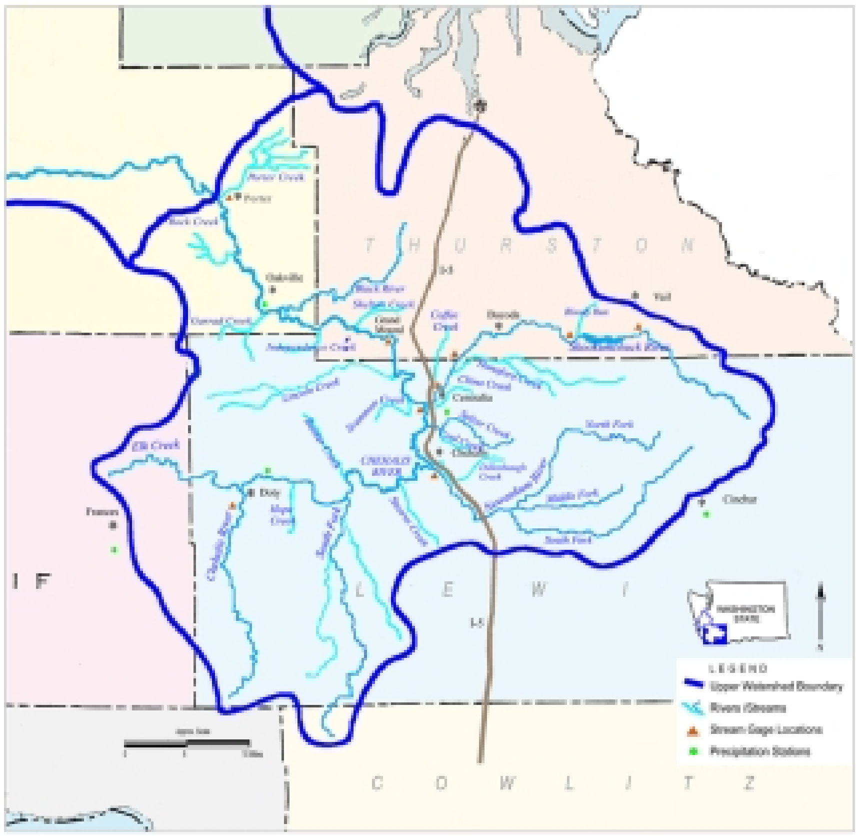


Figure 2-2: Upper Chehalis River Basin Boundary

Skookumchuck River

The Skookumchuck River, one of the major Chehalis River tributaries, joins the Chehalis River at RM 67, and is approximately 41 miles in length. It originates in the Mt. Baker-Snoqualmie National Forest northeast of the City of Centralia, and empties into the Chehalis River at Centralia. The total drainage area for the Skookumchuck River is 181 square miles. Elevations within the basin range from 150 feet at the mouth to 3,800 feet at the headwaters, with approximately two-thirds of the basin located below an elevation of 1,000 feet. The slope of the Skookumchuck River from its source to the town of Bucoda is steep, falling an average of 19 feet per mile. Below Bucoda, the slope flattens to about 5 feet per mile near Centralia. Except for the uppermost portion, the Skookumchuck River flows as a meandering channel in a floodplain, varying in width from a few hundred feet to 0.5 mile.

The Skookumchuck River Basin has three distinctly different hydrologic regions of approximately the same size. The region above Bloody Run Creek has a drainage area of 66 square miles and is a steep, well-forested, mountainous area with elevations generally above 1,000 feet. The river in this region flows through a steeply sided, narrow floodplain that drains into the Skookumchuck Reservoir. The region from Bloody Run Creek to the mouth (excluding the Hanaford Creek drainage) has a drainage area of 56 square miles and contains a relatively broad (.5 to 1 mile wide) floodplain bordered by steeply sided ridges. Hanaford Creek drains into the Skookumchuck at RM 3.8 and has a drainage area of 59 square miles. Hanaford Creek is broad and is at relatively low elevations with a substantial amount of natural overbank storage compared to the mainstem.

Three developments are notable within the Skookumchuck River system. The first is the City of Centralia, which occupies several square miles at the lower end of the basin. The second development is Skookumchuck Dam, located about 20 miles upstream from Centralia and operated by PacifiCorp. Skookumchuck Dam was completed in 1971 and has been considered several times for flood control use. The third development of note in the Skookumchuck Basin is the Centralia Steam Generating Plant on Hanaford Creek. Authority has been granted for this coal-fired facility to divert up to 54 cubic feet per second (cfs) of water from the Skookumchuck River.

The Skookumchuck River is regulated by the Skookumchuck Dam, which is owned by Scottish Power (PacifiCorp). Skookumchuck Dam is located at RM 21.9, just upstream from Bloody Run Creek. The dam is an earthfill structure approximately 190 feet high with a crest elevation of 497 feet. Construction of the dam was completed in January 1971. The primary purpose of the project is water supply for the Centralia coal-fired power generator plant. Outflow from the reservoir is either over the spillway crest at elevation 477 feet or through the outlet works with intake gates at elevations 449, 420, and 378 feet. The discharge capacity of the outlet works is approximately 220 cfs when the pool elevation is at the spillway invert. Because of this limited outlet capacity, the reservoir typically fills early in the flood control season and passes subsequent floods over the 28,000 cfs capacity spillway. The normal active storage capacity of the reservoir is 38,700 acre-feet (ac-ft) between elevations 400 feet (normal minimum operating pool) and 492 feet (maximum operating pool). Additional usable storage of 3,170 ac-ft is available between elevations 378 feet (invert of the lowest intake) and 400 feet. Dead storage is approximately 1,420 ac-ft between elevations 378 and 340 feet.

The land use in the Skookumchuck River floodplain is generally agricultural in the upper reaches with increasing urbanization towards the mouth. The most developed portion of the floodplain is from the mouth to RM 4.5 with the city of Centralia’s central residential/business district being within the floodplain on the left bank near RM 2.0. The small town of Bucoda is within the floodplain on the right bank near RM 12.

Newaukum River

The Newaukum River joins the Chehalis River at RM 75 at the City of Chehalis. The Newaukum drains 175 square miles of lowland and foothills southeast of the City of Chehalis. Elevations in the basin range from approximately 180 feet at the confluence with the Chehalis River, to just over 3,000 feet in the upper basin. The Newaukum River is the second major tributary to the Chehalis River in Lewis County.

The Newaukum River is made up of three forks: the north, middle, and south forks. Upstream sections on both the north and middle forks have slopes of 83 feet per mile; the south fork has a slope of 188 feet per mile above the town of Onalaska. The average channel slope for the entire drainage is 35 feet per mile. The lower two miles of the stream and floodplain are within the flood backwater area of the Chehalis River.

South Fork Chehalis River

The South Fork Chehalis River joins the mainstem Chehalis River at RM 86 and drains 130 square miles. The lower basin (up to RM 9) consists of a broad, flat valley with small creeks draining the hills on either side. From RM 9 to RM 15, the valley narrows from 1.5 miles wide to 0.75 miles wide.

Upper Chehalis River above Doty

The upper Chehalis River is at comparatively lower elevations with most areas ranging in elevation from 200 feet to 1000 feet above sea level. The stream slope averages 16 feet per mile.

Coffee Creek

Coffee Creek is a tributary of the Skookumchuck River. With headwaters in Thurston County, Coffee Creek flows south through the Zenkner Valley to the Skookumchuck River north of Centralia. The watershed encompasses 6.2 square miles of moderately sloping hills. Watershed elevations range from 186 feet at the confluence with the Skookumchuck River to 645 feet at the northern tip of the watershed. The stream gradient is low in the lower four miles of the watershed. Coffee Creek has been moved from its natural location to a periphery channel bordering the edge of adjacent hills and the valley floor.

China Creek

China Creek is a relatively small, short stream that flows through the City of Centralia to the Chehalis River. The watershed extends about five miles east of the Chehalis River at Centralia. It encompasses approximately 4.4 square miles, ranging in elevation from 180 feet to 570 feet. Much of the land is moderately steep. Most of the channel consists of pipes and culverts through Centralia.

Salzer Creek

Salzer Creek flows into the Chehalis River from the east, just south of the Centralia city limits, and drains 24.3 square miles. Salzer Creek originates in the low-lying hills east of Centralia-Chehalis, and has a maximum elevation of about 800 feet. The stream gradient of Salzer Creek is relatively flat. The lower two miles of the stream are within the flood backwater area of the Chehalis River. Coal Creek, a major tributary of Salzer Creek, has a drainage area of 5.4 square mile, and a steeper slope.

Dillenbaugh Creek

Dillenbaugh Creek flows into the Chehalis River from the east, at the City of Chehalis. It originates in the steep foothills southeast of Chehalis, and has a drainage area of approximately 11.7 square miles. The gradient of Dillenbaugh Creek in the upper reaches is approximately 70 feet per mile. After it flows out onto the Newaukum River floodplain, the gradient drops as Dillenbaugh Creek parallels the Newaukum and Chehalis Rivers for nearly three miles before finally flowing into the Chehalis River. Dillenbaugh Creek collects much of the storm drainage from the City of Chehalis in this lower reach. Substantial flood flows overtopping the Newaukum River also enter into lower Dillenbaugh Creek.

Upper Chehalis River

The upper Chehalis River, above the Newaukum River, drains an area of 445 square miles, and can be divided into two main drainages and several smaller subdrainages. The two main drainages are the South Fork Chehalis River and the mainstem of the Chehalis River. The South Fork Chehalis River joins the mainstem of the Chehalis River at RM 88 and drains 130 square miles. The mainstem of the Chehalis River above Doty drains 113 square miles at RM 101.8 (USGS Gage). The major subdrainages include Bunker Creek, Stearns Creek, and Elk Creek, which drain 34.1, 34.8, and 46.7 square miles, respectively.

Centralia-Chehalis Reach

This reach of the river stretches from the Skookumchuck River at RM 66.89 to the Newaukum River at RM 75.20. This reach is comprised primarily of the Centralia-Chehalis floodplain, with both cities located within the reach. Dillenbaugh Creek, Salzer Creek, and China Creek all enter the Chehalis River along this reach. The river is characterized by a very shallow gradient and a meandering stream course in this area.

Grand Mound Reach

This reach of the river stretches from the upstream end of the Chehalis Indian reservation at RM 53, to the mouth of the Skookumchuck River at RM 66.89. The major subdrainages are Lincoln Creek and Scatter Creek, which drain 42.84 and 41.3 square miles, respectively. The Chehalis River drains 895 square miles at the Grand Mound gage.

Independence Creek

Independence Creek flows northeast out of the Doty Hills to enter the Chehalis River at about RM 51.07, immediately due south of the Chehalis Indian Reservation. Independence Creek extends over 8 miles into the watershed and drains approximately 26 square miles. The watershed ranges in elevation from 630 feet to 105 feet at the confluence with the Chehalis River. Much of the land is steep hillside with a small half-mile wide valley along the bottom.

Black River

The Black River is located in the west central portion of the Chehalis River Basin and is characterized by relatively flat topography. The Black River originates in Black Lake, about three miles west of Tumwater, and is about 25 miles in length. The river flows generally southwest and begins to meander in the downstream portion where it flows just north of the Chehalis Indian Reservation. The Black River drains approximately 136 square miles at its mouth. Significant amounts of flood flow overtopping the Chehalis River right bank and crossing the floodplain and State Route (SR) 12 within and east of the Reservation enter into the Black River between RM 5 and RM 9.

Porter Reach

The floodplain in this reach is approximately 1.5 to 2 miles wide and is confined between the Black Hills to the north and the Doty Hills to the south. The principal communities within the reach include the cities of Porter and Oakville, and the Chehalis Indian Reservation. Porter, Oakville, and other small rural communities are generally situated on the floodplain margins. The Chehalis Indian Reservation lies directly within the floodplain with development generally occupying the limited areas of high ground. Major subdrainages include Garrad Creek, Rock Creek, Shelton Creek, and Porter Creek, which drain 26, 24.8, 35.9, and 35.3 square miles, respectively. The Chehalis River drains 1,294 square miles at the Porter gage.

2.2 GEOLOGY AND SOILS

2.2.1 Geology

The bedrock geology of the Chehalis River Basin is composed primarily of igneous and sedimentary bedrocks of the Tertiary Period. Surficial deposits include the unconsolidated glacial sediments of the Pleistocene Epoch. Following formation of the bedrock 7 to 55 million years ago, the area underwent geologic uplift, raising the volcanic and sedimentary rocks above sea level. Deformation, in the form of faulting and folding, accompanied the uplift. Landslides, erosion, glaciation, and glaciofluvial deposition, as well as recent volcanic activity, followed. The most recent 10,000 years have been a period of relatively stable climatic and geologic conditions with erosion being the dominant geologic process (ENSR 1994).

From the City of Chehalis to the City of Montesano, the average width of the floodplain is about 1.5 to 2.0 miles. The sediments within this floodplain attain a maximum depth of approximately 100 feet. The floodplain shows very little relief, either longitudinally or perpendicular to the direction of flow. This lack of relief has resulted in a very sinuous river course with numerous oxbow lakes and other abandoned channels.

Geologic evidence indicates that the Chehalis River has reworked its valley since the deposition of the glacial alpine outwash sand and gravel. This sand and gravel forms the older river terraces that line the valley margins. This timeline would make the recent river deposits less than 7,000 to 10,000 years old. Canyon wall conditions imply a mature topographic landscape prior to river sedimentation. This type of landscape would contribute to the long-term, slow aggradation by the river system with deposition of fine sand and some fine gravel, but a predominance of silt, clay, and organic mud. Mapping of the Centralia-Chehalis area by the Soil Conservation Service (SCS) confirms that at least 50 percent of the deposits in the upper 5 feet of the valley sediments are organic mud, silt, and plastic clay. The longer-term, more active stream channels contain coarser grained sediments.

2.2.2 Soils and Vegetation

The SCS published a soil survey of Lewis County in May 1987. Much of the information in this section is excerpted from that document (SCS 1987). Soils in the floodplain tend to be a silty clay loam. These soils tend to be very deep and range from poorly drained to well drained. The native vegetation is wetland plants, deciduous plants, and conifers. The common wetland plants include bull thistle, cattail, peachleaf willow, reed canarygrass, and soft rush. The main woodland species are Douglas fir and red alder. Among the trees of limited extent are black cottonwood, western red cedar, and bigleaf maple. Among the common forest understory plants are western swordfern, vine maple, cascade Oregon grape, red huckleberry, western brackenfern, Pacific trillium, and trailing blackberry.

Soils on plains, terraces and uplands tend to be very deep, and range from well-drained gravelly sand to poorly drained silty clay. The main woodland species are Douglas fir and red alder. Other trees found in limited quantities are western hemlock, western red cedar, and bigleaf maple. Among the common forest understory plants are cascade Oregon grape, rose, red huckleberry, western brackenfern, violet, and salal.

Soils on uplands, mountains, and high terraces tend to be very deep, well-drained silt loam. The main woodland species are Douglas fir and red alder. Other trees found in limited quantities are western hemlock, western red cedar, and bigleaf maple. Among the common forest understory plants are cascade Oregon grape, salmonberry, red huckleberry, western brackenfern, vine maple, and red elderberry.

All soils in the basin fall predominately within AASHTO hydrologic group A. Soil permeability typically ranges from 0.6 to 2 inches per hour.

2.3 CLIMATE

The Centralia-Chehalis area has a predominately marine climate characterized by mild temperatures both summer and winter. Extreme temperatures are unusual for the area because prevailing westerly winds bring maritime air over the basin and provide a moderating influence throughout the year.

During the spring and summer, high-pressure centers predominate over the northeastern Pacific, sending a northwesterly flow of dry, warm air over the basin. The dry season extends from late spring to midsummer, with precipitation frequently limited to a few light showers. Average summer temperatures are in the 50s or 60s (degrees Fahrenheit), but occasionally hot, dry easterly winds cross the Cascade Mountains and raise daytime temperatures into the 90s. The Aleutian low-pressure center normally predominates during the winter, causing a counterclockwise circulation of cool, moist air over the basin and prevailing southwesterly winds.

The area from the Pacific Ocean to the crest of the Olympic Mountains, the western slopes of the Cascade Range, and the Black and Willapa Hills receives the full force of winter storms. Virtually every fall and winter (October through March), strong winds and heavy precipitation occur throughout the basin. Storms are frequent and may continue for several days. Successive secondary weather fronts with variable rainfall, wind, and temperatures may move onshore at daily intervals or less. Heavy orographic-type rainfall is frequently produced by these storm conditions when warm, maritime, saturated winds rise over the coastal range and west slopes of the Cascade Range. Occasional short cold periods are experienced when movement of arctic air into the Northwest interrupts the usual weather pattern.

The locations of the climatological stations in the region are shown in Figure 2-1. A summary of pertinent data for these stations is shown in Table 2-1. The first eleven stations listed are all National Weather Service (NWS) stations, and the final station is at the Centralia Steam Plant where climatological data is collected by plant operators.

Table 2-1: Climatological Stations and Data Summary

Station Name	NWS Station ID	Data Type	Eleva-tion	Avg. Annual Precip. (in.)	Period of Record
Aberdeen	8	Daily	10	58.5	1931-Present
Aberdeen 20 NNE	13	Daily & Hourly	435	130.29	1948-Present
Centralia 1W	1277	Hourly	185	41.64	1931-Present
Chehalis	1330	Hourly	180	40.62	1948-1968
Cinebar 2E	1457	Hourly	1040	72.44	1948-Present
Doty 3E	2220	Daily	260	51.91	1978-Present
Elma	2531	Daily	69	66.83	1948-Present
Frances	2984	Hourly	231	71.91	1948-Present
Montesano 1S	5549	Hourly	25	76.79	1954-Present
Oakville	6011	Daily	80	56.06	1948-Present
Olympia AP	6114	Daily & Hourly	165	50.24	1948-Present
Centralia Steam Plant	N/A	Daily	200	49.72	1968-Present

Source: National Weather Service and PacifiCorp

Precipitation in the basin is affected by distance from the Pacific Ocean, elevation, and seasonal conditions. Generally, the southern slopes of the Olympic Range and the more easterly, higher slopes along the Cascade Range receive the greatest precipitation. The Black Hills in the northeast portion of the basin and Willapa Hills between the coast and the Centralia-Chehalis area often receive moderate to heavy rainfall during the movement of oceanic storms through the basin.

The greatest amount of rainfall occurs between the months of October and March. The abundance of rainfall during this period is due to the frequent storm systems that pass over western Washington. In Centralia, monthly rainfall totals for this period typically range between 5 and 8 inches. For the rest of the year, average monthly rainfall totals range only between 0.8 and 2 inches. The month with the highest average rainfall is November, with 7.77 inches. The month with the lowest average rainfall is July, with only 0.84 inches. Annual precipitation averages 41.64 inches, with a record low of 28 inches and a record high of 60 inches.

Temperature variations in the Skookumchuck basin depend on elevation, season, and several climatological factors. The weather station at Centralia has recorded temperature extremes of 105 to –16 degrees. The mean monthly temperature is 52 degrees with the monthly means of January and July being 39 and 65 degrees respectively. The growing season (the average period between killing frosts) is about 180 days.

Snowfall in the region is not heavy, but potential does exist for extremely large amounts on occasion. The average annual snowfall is approximately nine inches, with recorded extreme annual maximums at 45 inches. Most of the snowfall occurs in the month of January, with the monthly average at about 4.5 inches.

Snowfall occurs occasionally at Centralia but warm temperatures typically limit any snow accumulation over prolonged periods. Very little of the Upper Chehalis basin is above 3,000 feet. Consequently, a significant snow pack generally does not build up even in the higher areas of the basin.

Winds in the region rarely exceed 30 mph; winds of this speed usually only occur during the fall and winter months in conjunction with rainstorms and/or thunderstorms that pass through the vicinity. Approximately 10 percent of the winds between the months of November and February have speeds between 15 and 30 mph, compared with approximately two percent of the winds for the other months. The rest of the wind speeds typically range between zero and 15 mph, about 90 percent of the time. Wind speeds have been measured in excess of 70 mph during the winter months. The majority of the highest wind speeds measured have originated from the south and southwest directions.

2.4 STREAM FLOW CHARACTERISTICS

2.4.1 Streamgage Stations

Figure 2-2 shows locations of the U.S. Geological Service (USGS) streamgage stations currently in operation in the Upper Chehalis River Basin. Table 2-2 summarizes pertinent data for these stations. In addition to the USGS streamgage stations, the NWS maintains wire weight stage gages at the Mellen Street and Pearl Street Bridges. The gages are used by the NWS for flood forecasting and warning.

The available streamgage records for the Upper Chehalis basin can also be found in Table 2-2. Chehalis River near Grand Mound, Newaukum River near Chehalis, and Chehalis River near Doty all have at least 55 years of record. Skookumchuck River near Vail, Skookumchuck River near Centralia, Skookumchuck River below Bloody Run Creek, Skookumchuck River near Bucoda, South Fork Chehalis River at/near Boistfort, and Chehalis River at Porter all have at least 30 years of record. These gages with extended records represent each of the four main subbasins discussed in section 2. Additionally, there are seven other gages on smaller streams to help identify the flow characteristics of the smaller streams.

2.4.2 Runoff

Stream flow generated within the Chehalis River Basin originates primarily from rainfall; although, snowmelt occasionally augments runoff in the highest elevation reaches of the basin. The average annual runoff of the Chehalis River at its mouth (drainage area 2,114 square miles) and at the USGS streamgage near Grand Mound (drainage area 895 square miles), are estimated to be 6.4 million ac-ft and 2.0 million ac-ft, respectively.

Flows in the rivers and creeks of the Chehalis River Basin show seasonal variation characterized by sharp rises of relatively short duration from October to March, corresponding to the period of heaviest rainfall. After March, the flows tend to gradually decrease to a relatively stable base flow, which is maintained from July into October.

Major flooding occurs during the winter season, usually from November through February, as the result of heavy rainfall occasionally augmented by snowmelt. Flooding may be either widespread throughout the Chehalis River Basin or localized in subbasins. Some storms may cover the entire basin and cause widespread flooding. Other storms may center over the Willapa Hills and cause flooding of the upper Chehalis River or center over the Black Hills and Cascade Foothills and result in flooding of the Skookumchuck and Newaukum Rivers.

Table 2-2: USGS Streamgage Information

Station Name	Station ID	Drainage Area (sq. mi.)	River Mile	Record Period
Chehalis River near Doty	12020000	113	101.8	1939-Present
Elk Creek near Doty	12020500	46.7	2.5	1942-1970
S.F. Chehalis River near Boistfort	12020900	44.9	8.0	1965-1980
S.F. Chehalis River at Boistfort	12021000	48	6.0	1942-1965
Chehalis River near Chehalis	12023500	434	77.5	1929-1931
M.F. Newaukum River near Onalaska	12024000	42.4	8.0	1944-1971
N.F. Newaukum River near Forest	12024500	31.5	6.5	1960-1966
Newaukum River near Chehalis	12025000	155	4.1	1929-1931 1942-Present
Salzer Creek near Centralia	12025300	12.6	3.9	1968-1971
Skookumchuck River near Vail	12025700	40	28.8	1967-Present
Skookumchuck River near Centralia	12026000	61.7	21.0	1929-1969
Skookumchuck River below Bloody Run Creek	12026150	65.9	20.7	1969-Present
Skookumchuck River near Bucoda	12026400	112	6.4	1967-Present
Lincoln Creek near Rochester	12027000	19.3	9.0	1942-1950
Chehalis River near Grand Mound	12027500	895	59.9	1928-Present
Chehalis River at Porter	12031000	1,294	33.3	1952-Present

Source: U.S. Geological Survey

2.4.3 Historical Floods

General

Precipitation totals at Centralia (Centralia 1W) for the 10 largest 1-day, 2-day, and 3-day storms of record are presented in Table 2-3. In comparison, the estimated 100-year 24-hour rainfall from the NOAA Atlas 2 varies in the basin from 4 inches in the Centralia area, to 8 inches in the higher elevation areas of the upper basin, and 12 to 13 inches in the headwaters of the Wynoochee drainage.

Table 2-3: Precipitation Totals Ranked for 10 Largest Storms at Centralia 1W

One-Day Storm		Two-Day Storm		Three-Day Storm	
Month & Year	Total Precip. (in.)	Month & Year	Total Precip. (in.)	Month & Year	Total Precip. (in.)
Jan. 1990	4.13	Nov. 1986	6.09	Nov. 1986	6.49
Nov. 1990	3.96	Dec. 1933	5.10	Feb. 1996	6.40
Dec. 1933	3.95	Feb. 1996	5.02	Jan. 1990	5.87
Nov. 1986	3.22	Jan. 1990	4.96	Dec. 1933	5.49
Oct. 1942	3.22	Nov. 1990	4.82	Dec. 1937	5.41
Feb. 1996	3.34	Nov. 1932	4.02	Nov. 1990	5.25
Feb. 1951	3.15	Feb. 1951	3.84	Nov. 1932	4.47
Nov. 1932	3.07	Oct. 1942	3.59	Feb. 1951	4.22
Dec. 1937	2.10	Dec. 1937	3.58	Oct. 1942	4.20
Jan. 1972	1.95	Jan. 1972	3.13	Jan. 1972	3.64

Source: USACE, 1997b

The greatest flood discharge on the Chehalis River in the Centralia-Chehalis area during the last 70 years occurred in February 1996. Table 2-4 summarizes the largest floods of record in the basin.

Table 2-4: Ten Largest Floods on the Chehalis, Skookumchuck, and Newaukum Rivers (Since 1971)

Gage	Chehalis River near Grand Mound			Skookumchuck River near Bucoda			Newaukum River near Chehalis		
Year	Stage (ft.)	Disch. (cfs)	Rank	Stage (ft.)	Disch. (cfs)	Rank	Stage (ft.)	Disch. (cfs)	Rank
Feb. '96	20.04	74,900	1	17.87	9,370	1	13.34	13,800	1
Apr. '91	17.66	42,800	7	16.82	7,860	5	12.07	9,210	7
Nov. '90	18.12	48,000	5	17.23	8,400	3	12.73	10,300	4
Jan. '90	19.34	68,700	2	17.33	8,540	2	12.75	10,400	3
Nov. '86	18.41	51,600	3	15.01	5,770	10	12.76	10,700	2
Dec. '77	16.79	36,500	10	16.18	7,170	6	12.49	10,300	5
Dec. '75	17.73	44,800	6	15.42	6,110	8	10.85	8,020	10
Jan. '74	16.88	37,400	9	15.30	5,950	9	11.17	8,440	8
Jan. '72	18.21	49,200	4	16.82	8,190	4	12.12	9,770	6
Jan. '71	17.29	40,800	8	15.82	6,630	7	11.99	8,390	9

Source: USACE, 1997b

Brief descriptions of the three most recent, largest floods in the Centralia-Chehalis area (the January 1990, November 1990, and February 1996 floods) are provided below. Descriptions for the two 1990 events came from USGS Open File Reports (Hubbard, 1991,1994), and the description for the 1996 event came from the USACE After Action Report (USACE, 1996a).

January 1990 Flood

The January 1990 flood was primarily the result of a series of back-to-back storms accompanied by heavy rainfall over the eight-day period of January 3-10. The heaviest rainfall occurred on the seventh day of the storm, January 9, causing extreme flooding because the rain fell on soils that were saturated from the preceding rainstorms.

The complex storm system included high winds and strong surges of precipitation. The Centralia climatological station recorded 8 inches of rain during the eight-day period. This eight day total precipitation represents 19 percent of the total average yearly precipitation recorded at the station. The most intense precipitation in the basin occurred near the headwaters of the Skookumchuck and Newaukum Rivers.

The surges in precipitation resulted in more than one flood peak in many of the rivers and creeks in the basin. The streams did not return to base flow between storm surges. The early precipitation saturated soils in the basin and added greatly to runoff potential when the heaviest rains arrived on January 9. Peaks of record, up to this event, were recorded at the following gaging stations: Chehalis River near Doty, Chehalis River near Grand Mound, and Chehalis River at Porter. These flood peaks were estimated at the time to be the 100-year flood.

November 1990 Flood

Above average precipitation in October and early November resulted in saturated soils that contributed to flooding potential when the major storm arrived during the period of November 21-25. Between the occurrences of a smaller storm in early November and the major storm, wet weather accompanied by cool temperatures continued and snow levels descended to about the 1,000-foot elevation. The Cascade Foothills averaged 6 inches at elevations of 1,000 to 2,000 feet, 12 inches at 2,000 to 3,000 feet, and 12 to 18 inches at 3,000 to 4,000 feet. The water content of the snow was generally 10 percent or higher. As a warm front moved through western Washington on Wednesday, November 21, snow changed to rain and temperatures rose. The warm front caused melting of snow up to elevations of 5,500 feet. Over the next three days, intense rain fell on drainages that were starting to swell from snowmelt runoff resulting in disastrous flooding. A cold front moved in from the north on November 26, 1990, lowered freezing levels, and diminished precipitation, finally ending the severe flooding.

February 1996 Flood

The February 1996 flood is the flood of record to date, on all the major drainages in the Chehalis River Basin. Several of the main ingredients for a major storm flood were in place by February 5. The ground throughout the basin was at or near saturation due to above average precipitation, during the preceding weeks. In addition, snow had recently fallen as low as 500 feet above sea level during a cold snap. Warm, moist subtropical air was also being transported from the Pacific Ocean into the Pacific Northwest and the freezing level in this subtropical air mass was well above 8,000 feet, which meant warm rains on the snow pack in the foothills.

A strong polar jet stream with core wind speeds in excess of 150 knots extended into the central and western Pacific. Storms fed upon the stream and this powerful jet sustained and strengthened the storms as they moved in from the eastern Pacific. At the same time, the atmosphere had formed a blocking pattern, causing the major troughs and ridges around the Northern Hemisphere to remain stationary. The Pacific Northwest was situated between a major trough to the west and a major ridge to the east, ideal conditions for enabling weather systems to be at maximum strength when they reached the area. The atmosphere remained in this general pattern for at least 96 hours, during which copious amounts of rain fell and large quantities of water in the existing snow pack were released into the rivers.

2.4.4 Flood Exceedance Frequency

USACE recently updated their flood frequency curves for the Chehalis River in the vicinity of Centralia (USACE, 1997b). USACE had previously published flood frequency curves for the Chehalis River for a 1980 Federal Emergency Management Agency (FEMA) report (ENSR 1994), and made revisions to the curves in 1989 (USACE 1992). Since 1980, there have been three floods of record, and several other major floods on the Chehalis River. USACE incorporated the data acquired after 1980 and recomputed the frequency curves. The recomputed frequency curves data, shown as years of recurrence interval, are listed in Table 2-5. The recomputed frequency curves are significantly higher than those published both in 1980 and 1989.

Table 2-6 shows a comparison of estimated flood recurrence intervals for the Chehalis River at Grand Mound, using frequency numbers computed by the USACE and used by FEMA on various occasions.

Table 2-5: Peak Discharge Frequency Data for Selected Locations

Location	2-Year Flow (cfs)	10-Year Flow (cfs)	25-Year Flow (cfs)	50-Year Flow (cfs)	100-Year Flow (cfs)
Chehalis near Grand Mound	25,000	43,800	55,000	64,300	74,300
Skookumchuck at Mouth	5,200	9,000	10,600	11,900	13,000
Skookumchuck at Pearl St.	4,800	8,450	10,100	11,300	12,500
Skookumchuck near Bucoda	3,900	6,900	8,300	9,300	10,400
Chehalis at Mellen St.	18,400	32,700	41,400	49,000	57,200
Chehalis above Salzer Creek	17,900	31,900	40,400	47,600	55,700
Newaukum near Chehalis	5,800	9,300	11,200	12,400	13,800

Source: USACE, 1997b

Table 2-6: Comparison of Flood Recurrence Intervals at Grand Mound

Year	Date	Maximum Flow (cfs) at Grand Mound Gage	Flood Recurrence Interval (years)		
			USACE (1998 update)	USACE (1989 update)	FEMA (1980-present)
1996	Feb. 6	73,900	100	400	600
1990	Nov. 25	48,000	15	30	35
1990	Jan. 10	68,700	70	250	400
1972	Jan. 21	49,200	15	30	35

2.5 HYDROLOGY OUTPUT REQUIREMENTS FOR ANALYSIS OF PROJECTS

In order to analyze the proposed Flood Damage Reduction alternatives, representative hydrographs were determined for a number of locations within the project study area. The hydrographs were developed for the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year storm events. The locations chosen are:

- Chehalis River at Doty
- Elk Creek at Mouth
- Chehalis River above SF Chehalis River
- South Fork Chehalis River below Stillman Creek
- SF Chehalis River below Lake Creek
- SF Chehalis River at Mouth
- Bunker Creek at Mouth
- Chehalis River above Sterns Creek
- Sterns Creek at mouth
- Newaukum River below confluence of NF and SF Newaukum Rivers
- Newaukum River at gaging station (near Chehalis)
- Newaukum River at mouth
- Salzer Creek at RM 2.9
- Salzer Creek at mouth
- Chehalis River at Mellen Street bridge
- Skookumchuck River at Vail gaging station
- Skookumchuck River at Skookumchuck Dam
- Skookumchuck River at Bloody Run gaging station
- Skookumchuck River at Bucoda Gaging Station
- N. Hanaford Creek at approx. 2.8 miles above confluence with S. Hanaford Creek
- N. Hanaford Creek above S. Hanaford Creek
- S. Hanaford Creek at mouth
- Skookumchuck River at mouth
- Lincoln Creek below Sponenbergh Creek
- Lincoln Creek at mouth
- Chehalis River at Grand Mound gaging Station
- Chehalis River below Scatter Creek
- Chehalis River above Black River
- Black River at RM 11.1

- Black River at mouth
- Chehalis River at Porter gaging station

Pacific International Engineering (P.I. Engineering; PIE) has developed a UNET hydraulic model for the Upper Chehalis River Basin (see section 3 for more information about this model). The model requires input hydrographs, which is what the USACE was tasked to provide. The following is a list of locations where hydrographs were developed for input to the UNET model:

- Chehalis River at Doty gaging station
- Elk Creek at Mouth
- Hope Creek at Mouth
- South Fork Chehalis River at RM 5.84
- Stillman Creek at Mouth
- Lake Creek at Mouth
- Bunker Creek at Mouth
- Sterns Creek at RM 4.62
- Newaukum River at Chehalis gaging station
- Dillenbaugh Creek at RM 3.45
- Salzer Creek at RM 5.21
- Coal Creek at Mouth
- China Creek at Mouth
- Hanaford Creek at RM 6.28
- Packwood Creek at Mouth
- North Hanaford Creek at Mouth
- South Hanaford Creek at Mouth
- Skookumchuck Dam Outflow
- Bloody Run Creek at Mouth
- Johnson Creek at Mouth
- Thompson Creek at Mouth
- Salmon Creek at Mouth
- South Side of Bucoda Tributary at mouth
- Connor Creek at Mouth
- Coffee Creek at Mouth
- Lincoln Creek at RM 3.9
- Scatter Creek at Mouth

- Independence Creek at Mouth
- Black River at RM 11.1
- Garrard Creek at Mouth
- Shelton Creek at Mouth
- Rock Creek at Mouth
- Porter Creek at Mouth

Some of these locations were chosen because they are either located at a stream gage location, or they are located immediately upstream of areas where flood flows backwater. The upstream areas were chosen because PIE's UNET model takes into account the backwatering in these downstream areas and developing the hydrographs in these downstream areas would be more difficult than running the upstream areas through the UNET model.

2.6 HYPOTHETICAL HYDROGRAPH DEVELOPMENT METHOD

To determine the hypothetical flood hydrographs for each of the locations, the locations were broken down into full gage record sites, partial gage record sites, minimal gage record sites and ungaged sites. The fully gaged sites were defined as sites where there are lengthy historical records (greater than 50 years), and fairly continuous records through to the present. Gaged sites that either lacked data for the recent past (1990s), or that did not exist 40 years ago were considered partial gage records. This distinction is made because there have been several large events in the last 10 years, so gage records that did not include this data would under predict the flow for the basin and gage records that did not contain a significant record preceding the last 10 years of record would over predict the flow for the basin. Minimal gage record sites are sites with small records that could be used to provide rough flow comparisons between better gaged sites, but are difficult to use for full frequency analysis. Ungaged sites are areas where there are no gage records at all for the site.

2.6.1 Flood Frequency Analysis

For the fully gaged sites, the program HEC-FFA was used to perform the flood frequency analysis. This program computes flood frequencies in accordance with the publication titled "Guidelines for Determining Flood Flow Frequencies, Bulletin 17B of the US Water Resources Council". The flood frequency is determined by fitting a Log-Pearson Type III distribution to the data, and then making an expected probability adjustment. A generalized skew of 0.3 was used for the analysis of the peak events and a generalized skew of 0 was used for the 1-day, 3-day, 7-day, and 15-day analyses. With the fully gaged sites, the adopted skew used by the program is close to the actual skew of the data due to the long length of records at these sites. The sites that are considered to be fully gaged sites are: Chehalis River near Doty, Newaukum River near Chehalis, Chehalis River near Grand Mound, and Chehalis River at Porter. The results are through water year 1998, and the 2- to 500-year recurrence flows are listed in Table 2-7.

To perform a fully representative frequency analysis for the partially gaged sites, a two-station comparison is made to a fully gaged site that has similar characteristics. The two-station comparison is performed using the methodology outlined in the USACE manual EM 1110-2-1415 (1993) titled “Hydrologic Frequency Analysis”. These characteristics include drainage basin area, basin aspect, and basin elevations. This analysis indicates that the Upper Chehalis River above Doty, the South Fork Chehalis River, the Newaukum River, and Skookumchuck River have similar characteristics. The computed degree of correlation between each of the partially and fully gaged sites can be seen in Table 2-8. The fully gaged site’s skew was used for the two-station comparison. The 2- to 500-year recurrence flows are listed in Table 2-9.

An additional partially gaged site was developed by subtracting the Skookumchuck River near Bucoda gage from the Skookumchuck near Bloody Run gage. This site is called Bucoda Local. This was done because both gages by themselves are influenced by the regulation of the Skookumchuck Dam but the difference of the two is the unregulated flow between the two gage sites. This provides a partially gaged site for the lower Skookumchuck basin with a record of 30 years and includes the recent high flow events.

Two-station comparisons were not made with the minimal gage record sites because the limited data either did not result in good correlations with the fully gaged sites and/or the record did not provide a good representation of both high and low flows.

2.6.2 Correlation to Chehalis River at Grand Mound Gaging Station

Since the Grand Mound stream gage is the closest stream gage with a full record to the main damage center in the Centralia-Chehalis area, it was decided that the model should contain flows that represent the appropriate recurrence period at Grand Mound. This means that if a sub-basin of the Upper Chehalis has a pattern of running at a different recurrence interval than the one that Grand Mound is at, that recurrence flow would be the one that was input into the model. A correlation, therefore, was developed for each of the 8 recurrence intervals. This was done by setting a recurrence interval for each year’s peak event at Grand Mound, and then setting a recurrence interval for the same event at each subbasin. A best-fit line was then set to the data and the flows for the 8 recurrence intervals were extracted from that relationship. These relationships are shown in Table 2-10. These flows are the target volumes for the different duration events when developing the hypothetical hydrographs. The fully and partially gaged sites account for roughly 2/3 of the Grand Mound 1-day peak volume and over half of the longer duration peak volumes.

2.6.3 Shaping of the Hypothetical Hydrographs

Once these target volumes were established, an hour-by-hour hypothetical hydrograph could be developed. There are five fairly recent events (1/72, 1/90, 11/90, 12/94, 2/96) in which there is hourly data for the Chehalis River near Doty, Newaukum River near Chehalis, Skookumchuck River near Vail, Chehalis River near Grand Mound, and Chehalis River at Porter gage sites. These five events represent a broad range of recurrence intervals at the Chehalis River near Grand Mound site. Table 2-11 shows the recurrence intervals of the volumes observed at the Chehalis River near Grand Mound site for these five events.

Table 2-7: Expected Flood Frequency Discharges for Peak, 1-day, 3-day, 7-day, and 15-day Events for Fully Gaged Sites

Peak Recurrence (yrs)	Chehalis River near Doty Flow (cfs)	Newaukum River near Chehalis Flow (cfs)	Chehalis River near Grand Mound Flow (cfs)	Chehalis River at Porter Flow (cfs)
2	9,690	5,900	25,100	29,400
5	14,300	7,980	35,900	40,300
10	17,800	9,330	43,900	48,000
25	23,333	11,167	56,200	59,300
50	27,000	12,300	64,200	66,500
100	31,600	13,500	74,100	75,200
200	36,700	14,800	85,000	84,500
500	44,500	16,500	101,000	97,800
1-day				
2	6,760	4,620	23,700	28,500
5	9,420	6,170	32,900	38,200
10	11,400	7,230	39,400	44,500
25	14,500	8,783	49,167	53,200
50	16,500	9,750	55,300	58,400
100	19,000	10,900	62,700	64,300
200	21,800	12,100	70,600	70,400
500	25,900	13,800	81,900	78,600
3-day				
2	4,880	3,580	20,800	26,400
5	6,400	4,740	27,900	34,300
10	7,430	5,520	32,400	38,900
25	8,900	6,620	38,533	44,900
50	9,820	7,300	42,200	48,300
100	10,900	8,090	46,300	52,100
200	12,000	8,910	50,500	55,700
500	13,600	10,100	56,000	60,300
7-day				
2	3,510	2,650	16,300	22,100
5	4,440	3,440	21,400	27,900
10	5,000	3,930	24,400	31,200
25	5,710	4,583	28,200	35,433
50	6,130	4,970	30,400	37,900
100	6,580	5,400	32,700	40,400
200	7,020	5,840	35,000	42,900
500	7,590	6,410	37,900	46,100
15-day				
2	1,370	2,020	12,300	17,100
5	1,680	2,570	15,900	21,700
10	1,860	2,910	18,000	24,500
25	2,100	3,347	20,500	27,767
50	2,427	3,600	21,900	29,700
100	3,140	3,870	23,400	31,600
200	3,450	4,140	24,800	33,500
500	3,730	4,490	26,600	35,800

Table 2-8: Two Gage Comparisons of Partial Gage Records to Full Gage Records

Partially Gaged Site Name	Record Length (yrs)	Fully Gaged Site Name Comparison	Peak Degree of Correlation (R^2)	Equivalent Record Length (yrs)	1-day Degree of Correlation (R^2)	Equivalent Record Length (yrs)	3-day Degree of Correlation (R^2)	Equivalent Record Length (yrs)	7-day Degree of Correlation (R^2)	Equivalent Record Length (yrs)	15-day Degree of Correlation (R^2)	Equivalent Record Length (yrs)
South Fork Chehalis River at/near Boistfort	25*	Chehalis River near Doty	0.77	51	0.8	46	0.83	48	0.89	51	0.89	51
Skookumchuck near Vail	30	Newaukum near Chehalis	0.8	50	0.79	48	0.78	48	0.84	54	0.91	53
Skookumchuck near Centralia	28	Newaukum near Chehalis	0.7	45	0.56	39	0.68	42	0.66	41	0.86	50
Skookumchuck-Bucoda Local**	27	Newaukum near Chehalis	0.32***	32***	0.77	46	0.78	46	0.77	46	0.78	47

* Peak record has 36 years of record
** Bucoda Local is the subtraction of the Skookumchuck near Bucoda gage from the Skookumchuck near Bloody Run gage
*** Peaks calculated by peak-1-day relationship not by actual data

Table 2-9: Expected Flood Frequency Discharges for Peak, 1-day, 3-day, 7-day, and 15-day Events for Partially Gaged Sites

Peak Recurrence (yrs)	Skookumchuck River near Vail Flow (cfs)	South Fork Chehalis at/near Boistfort Flow (cfs)	Skookumchuck River near Centralia Flow (cfs)	Skookumchuck River Bucoda Local Flow (cfs)
2	2402	3183	4040	*
5	3617	4618	5809	*
10	4460	5678	6996	*
25	5658	7297	8635	*
50	6393	8334	9628	*
100	7243	9605	10756	*
200	8111	10970	11892	*
500	9291	12937	13415	*
1-day				
2	1591	2514	2525	1324
5	2335	3502	3474	2082
10	2878	4225	4134	2664
25	3695	5320	5087	3581
50	4214	6021	5678	4178
100	4842	6876	6374	4922
200	5510	7793	7098	5733
500	6461	9111	8103	6917
3-day				
2	1261	1811	1818	972
5	1783	2278	2369	1426
10	2145	2581	2728	1749
25	2664	2994	3217	2225
50	2986	3242	3512	2523
100	3362	3523	3846	2877
200	3751	3807	4181	3248
500	4289	4187	4632	3767
7-day				
2	1022	1273	1315	688
5	1326	1583	1684	957
10	1515	1764	1912	1134
25	1760	1989	2205	1373
50	1904	2118	2377	1517
100	2062	2254	2563	1677
200	2216	2383	2745	1838
500	2415	2546	2980	2051
15-day				
2	765	909	965	489
5	963	1149	1258	678
10	1081	1288	1435	799
25	1227	1458	1646	958
50	1311	1555	1768	1052
100	1401	1655	1898	1155
200	1486	1750	2022	1256
500	1594	1868	2181	1387

* - Insufficient data to calculate

Table 2-10: Correlated Flows to Grand Mound

	Chehalis-Doty		Newaukum-Chehalis		Skookumchuck-Vail		SF Chehalis River- Boistfort		Skookumchuck-Centralia	
Peak Recurrence (yrs)	Associated Recurrence (yrs)	Chehalis-Doty Flow (cfs)	Associated Recurrence (yrs)	Newaukum-Chehalis Flow (cfs)	Associated Recurrence (yrs)	Skookumchuck-Vail Flow (cfs)	Associated Recurrence (yrs)	ehalis River- Bois Flow (cfs)	Associated Recurrence (yrs)	okumchuck-Centra Flow (cfs)
Relationship Equation*	$y=0.6537*x+1.9579$		$y=0.6722*x+1.3407$		$y=1.284*x^{1.0145}$		$y=1.0288*x^{0.9798}$		$y=1.0761*x^{0.7951}$	
Degree of Correlation (R^2)	0.89		0.83		0.82		0.72		0.62	
2	3.27	11634	2.69	6375	2.59	2642	2.03	3492	1.87	3817
5	5.23	14458	4.70	7773	6.57	3882	4.98	5032	3.87	5142
10	8.49	16746	8.06	8807	13.28	4732	9.82	6159	6.71	6216
25	18.30	20871	18.15	10365	33.63	5791	24.10	7635	13.91	7443
50	34.64	24185	34.95	11447	67.95	6698	47.53	8962	24.14	8344
100	67.33	28594	68.56	12745	137.27	7566	93.74	10316	41.88	9225
200	132.70	33268	135.78	13965	277.31	8415	184.88	11755	72.68	10140
500	328.81	40049	337.44	15579	702.54	9664	453.71	13796	150.59	11331
1-day										
Relationship Equation	$y=0.8888*x+0.501$		$y=1.119*x^{0.8172}$		$y=1.1083*x^{1.0049}$		$y=0.8717*x^{0.9805}$		$y=1.147*x^{0.836}$	
Degree of Correlation (R^2)	0.94		0.71		0.8302		0.68		0.44	
2	2.28	7007	1.97	4578	2.22	1647	1.72	2483	2.05	2540
5	4.95	9371	4.17	5741	5.59	2399	4.22	3545	4.40	3286
10	9.39	11158	7.35	6667	11.21	2946	8.33	4351	7.86	3852
25	22.72	13772	15.53	7822	28.15	3647	20.47	5466	16.91	4588
50	44.94	15994	27.37	8656	56.49	4296	40.38	6208	30.19	5092
100	89.38	18469	48.22	9664	113.36	4932	79.68	7130	53.90	5732
200	178.26	21191	84.96	10554	227.49	5597	157.23	8082	96.21	6321
500	444.90	25147	179.65	12732	571.28	6572	386.11	9404	206.97	7121
3-day										
Relationship Equation	$y=1.1145*x^{-0.0817}$		$y=0.7649*x+0.6664$		$y=1.0521*x^{1.0013}$		$y=0.7247*x^{1.2558}$		$y=1.0869*x^{0.9639}$	
Degree of Correlation (R^2)	0.95		0.94		0.87		0.80		0.61	
2	2.15	4955	2.20	3656	2.11	1280	1.73	1839	2.12	1840
5	5.49	6501	4.49	4543	5.27	1802	5.47	2519	5.13	2378
10	11.06	7537	8.32	5257	10.55	2164	13.06	2915	10.00	2728
25	27.78	8798	19.79	6264	26.41	2606	41.28	3422	24.19	3132
50	55.64	9942	38.91	6923	52.87	3007	98.57	3839	47.19	3471
100	111.37	11025	77.16	7729	105.84	3385	235.37	4206	92.04	3793
200	222.82	12122	153.65	8530	211.87	3773	562.07	4612	179.54	4112
500	557.17	13905	383.12	9636	530.32	4315	1776.33	5392	434.24	4533
7-day										
Relationship Equation	$y=0.9255*x^{1.0941}$		$y=0.955*x^{1.0362}$		$y=0.975*x^{1.1311}$		$y=0.8301*x^{1.0638}$		$y=1.0873*x^{1.0251}$	
Degree of Correlation (R^2)	0.802		0.82		0.82		0.7728		0.69	
2	1.98	3485	1.96	2639	2.14	1036	1.74	1290	2.21	1341
5	5.38	4483	5.06	3446	6.02	1364	4.6	1684	5.66	1714
10	11.49	5075	10.38	3947	13.19	1570	9.61	1911	11.52	1943
25	31.32	5738	26.83	4522	37.17	1812	25.48	2141	29.47	2200
50	66.87	6282	55.01	5013	81.42	2003	53.27	2323	59.97	2414
100	142.75	6768	112.82	5456	178.32	2182	111.36	2478	122.05	2603
200	304.74	7219	231.38	5900	390.57	2343	232.79	2622	248.39	2783
500	830.46	8218	597.97	6596	1101.06	2595	617.01	2811	635.42	3027
15-day										
Relationship Equation	$y=1.012*x^{1.0031}$		$y=0.9219*x^{1.0576}$		$y=1.1169*x^{0.9478}$				$y=1.0417*x^{1.0542}$	
Degree of Correlation (R^2)	0.7622		0.84		0.85		0.84		0.67	
2	2.03	2595	1.92	1970	2.15	775	1.84	945	2.16	981
5	5.09	3145	5.06	2574	5.13	966	3.88	1157	5.68	1282
10	10.19	3455	10.53	2926	9.90	1078	6.82	1310	11.80	1462
25	25.55	3789	27.74	3318	23.60	1200	14.38	1465	31.01	1652
50	51.22	4056	57.75	3642	45.53	1293	25.28	1568	64.39	1805
100	102.66	4286	120.19	3925	87.82	1379	44.44	1669	133.70	1940
200	205.75	4495	250.18	4199	169.41	1460	78.13	1760	277.64	2063
500	515.84	4774	659.35	4676	403.74	1559	164.69	1875	729.45	2234
Method										

* y represents the gage site recurrence interval and x represents the Grand Mound recurrence interval
** - Insufficient data to calculate
*** - Does not include Skookumchuck - Bucoda Local or Skookumchuck - Vail because locations are not used directly

Table 2-11: Recurrence Intervals in Years of Flow Volumes for Chehalis near Grand Mound Gage

	12/94	11/90	1/72	1/90	2/96
Peak	5.0	13.3	14.7	64.7	103.8
1-day	5.5	14.9	20.1	75.8	134.4
3-day	4.1	7.4	18.7	32.9	104.9
7-day	3.3	3.1	11.5	12.1	29.0
15-day	6.5	3.5	7.7	4.2	6.0

The hypothetical hydrographs were shaped to match the shape of the observed event of the same recurrence at the gage site (i.e., 5-year hypothetical to 12/94 observed event). For the recurrences that do not have a matching observed event, the next closest event is chosen to shape from. This shape was smoothed to represent a more typical average condition. The flow volumes from early hydrograph humps due to an initial surge of runoff from impervious surfaces were accounted for in the smoother upward rising limb. These hydrographs are then adjusted to ensure they match the needed volumes for all of the time intervals (peak, 1-day, 3-day, 7-day, 15-day). Priorities were set to ensure that the peak and 1-day volumes are most accurate (+/- 1 percent), with the greater volume discrepancies being found in the longer durations (3-day is +/- 5 percent, and the 7- and 15-day are +/- 10 percent).

Base flow for each of the gaged sites was determined by examining the days surrounding the yearly peak in the gage records and selecting a base flow before the start of each of these events. The average of each of these peak event base flows is the base flow that was used for all of the hypothetical hydrographs for that gage location.

2.6.4 Development of Hydrographs for Minimally Gaged and Ungaged Sites

The sites where the flow records are not substantial enough for frequency analysis were sorted into basins with similar characteristics. The characteristics used were: aspect of the basin, drainage area, stream discharge per square mile of drainage area, and proximity to the mainstem Chehalis River. The knowledge that certain basins correlated well with others in the two-station analysis was used to further categorize the minimally gaged and ungaged sites. Once the sites were categorized, the minimal gage records were used to see if the observed flows match the flows derived from the categorization.

An analysis of discharge per square mile at each gage site was done to find relationships that could be used to scale the gaged hypothetical hydrographs to make ungaged hypothetical hydrographs. A ratio of discharge per square mile of basins whose drainages are close to the mainstem Chehalis (Elk Creek, Salzer Creek, Bucoda Local, Newaukum Local, Porter Creek, Rock Creek) to the upper basins which draw from basin areas that are further removed from the mainstem (Chehalis River near Doty, South Fork Newaukum River, Skookumchuck near Vail, Newaukum River near Chehalis, Skookumchuck River near Centralia) shows that these lower drainages have less runoff (see Table 2-12).

Table 2-12: Ratios of 1-day Discharge per Square Mile from Lower Basins to Upper Basins

Lower Basin to Upper Basin	Ratio of Discharge per Square Mile	Standard Deviation of Ratio	Years of Concurrent Record
Elk Creek/Chehalis River near Doty	0.44	0.11	5
Salzer Creek/Newaukum River near Chehalis	0.73	0.16	3
Bucoda Local/Skookumchuck River near Vail	0.73	0.22	29
Newaukum Local*/South Fork Newaukum River near Onalaska	0.72	0.16	18
Porter Creek/Skookumchuck near Centralia	0.75	0.11	4
Rock Creek/Skookumchuck near Centralia**	0.73	0.19	25
Black River/Skookumchuck near Centralia	0.39	0.06	6

* Newaukum Local was calculated by subtracting the South Fork Newaukum River near Onalaska gage from the Newaukum River near Chehalis record.
** Rock Creek/Skookumchuck near Centralia comparison is for peak flows because 1-day data is not available for Rock Creek.

These ratios were used to scale all of the ungaged basins that have drainage areas close to the mainstem Chehalis. The scaling factors are shown in Table 2-13.

Table 2-13: Flow Determination for Tributaries in Close Proximity to the Mainstem Chehalis River

Tributary Location	Ratio of Upper Basin Flows	Upper Basin Gage Used
Upper Chehalis River above South Fork Chehalis River	0.5	Chehalis River near Doty
South Fork Chehalis River down to Newaukum River	0.5	South Fork Chehalis at Boistfort
Newaukum River down to Skookumchuck River	0.7	Newaukum River near Chehalis
Skookumchuck River	1.0	Skookumchuck – Bucoda Local
Grand Mound to Porter excluding Black River	0.75	Skookumchuck near Centralia
Black River	0.4	Skookumchuck near Centralia

The limited gage record on Lincoln Creek shows that it has a similar discharge per square mile to both Newaukum and Bucoda Local. Additionally, Lincoln Creek and Bunker Creek have similar source locations and similar drainage areas, so hypothetical hydrographs for both are based off the Bucoda Local hypothetical hydrographs. When the flows for the larger events (100-, 200-, and 500-year) are routed downstream, the flow at Grand Mound is too high using these ratios. The only actual event that can provide a glimpse to how these ratios may differ in extreme events is the February 1996 event. The only discharge per square mile ratio from lower basins to upper basin that exists for that year is Bucoda Local to Skookumchuck near Vail, which has a ratio of 0.63. This is a tenth smaller than the average for the period of record. For these large events, a ratio of 0.4 and 0.6 was used for the Chehalis River near Doty/South Fork Chehalis River and Newaukum River areas.

2.6.5 Flood Timing

There are hourly records for five large events (1/72, 1/90, 11/90, 12/94, 2/96) at the Chehalis River near Doty, Newaukum River near Chehalis, Skookumchuck River near Vail, and Chehalis River near Grand Mound gage sites. As Table 2-11 shows, these events represent a good range of recurrence intervals. To ensure that the flows matched up correctly downstream, the timing was calculated based on the time of the gage peak in relation to the peak at the Chehalis River at Grand Mound gage site (see Table 2-14). A relation was made between the recurrence interval and the time to Grand Mound peak for these gaged sites. The December 1994 timing is an outlier at most sites so it was omitted in most of the relationships. Often the relationship broke down when evaluating the timing above a 100-year event, so a more reasonable timing was selected for those events.

P.I. Engineering set up HEC-1 models for each of the five events at all of the locations that are not represented by these gages. The timing of the HEC-1 runs for each of the basins was broken down into the same groupings (Chehalis River near Doty/South Fork Chehalis River, Newaukum River, and Skookumchuck River) as was done for the flow. An average of the timing for each of the subbasins was used to develop the recurrence versus time before Grand Mound peak relationship. The timing for all of the locations can be seen in Table 2-15.

2.6.6 Regulation of Skookumchuck Flows

The recurrence flows for Skookumchuck at Centralia represent the inflows to Skookumchuck Dam. To appropriately mimic existing conditions, this flow was routed through the dam to obtain a regulated outflow. This was done adapting a HEC-5 model that was developed in 1990, by Matt Johannson, for reservoir simulation of power loss. The reservoir elevation was assumed to start at the existing spillway crest height of 477 feet, as it usually is for most large events.

Table 2-14: Observed Flood Timing

Event Date/ Time to Peak (hrs)	Grand Mound Recurrence Interval (yrs)	Doty	Newaukum	Vail	Grand Mound
Feb-96	100	2/8/1996 14:00	2/8/1996 15:00	2/8/1996 14:00	2/9/1996 8:00
Time to Grand Mound (hrs)		18	17	18	0
Jan-90	65	1/9/1990 14:00	1/9/1990 20:00	1/9/1990 15:00	1/10/1990 12:00
Time to Grand Mound (hrs)		22	16	21	0
Jan-72	15	1/20/1972 18:00	1/21/1972 3:00	1/20/1972 18:00	1/21/1972 18:00
Time to Grand Mound (hrs)		24	15	24	0
Nov-90	13	11/24/1990 16:00	11/24/1990 22:00	11/24/1990 16:00	11/25/1990 19:00
Time to Grand Mound (hrs)		27	21	27	0
Dec-94	5	12/20/1994 9:00	12/21/1994 0:00	12/20/1994 19:00	12/21/1994 14:00
Time to Grand Mound (hrs)		29	14	19	0

Table 2-15: Chehalis River Subbasin Timing for Different Recurrence Intervals

Timing for Basins

(in hours that basin peaks prior to Grand Mound)

Recurrence (in years)	2	5	10	25	50	100	200	500
Skookumchuck-Vail	26	25	25	24	22	18	17	16
Tribs	29	27	26	25	24	23	22	21
Newaukum-Chehalis	15	15	15	15	16	16	16	17
Tribs	19	19	19	20	21	24	26	27
Chehalis-Doty	26	25	25	24	22	19	19	18
Tribs	29	27	26	24	23	22	20	19

3. BASELINE FLOOD MODELING

3.1 INTRODUCTION

To evaluate the potential effects of various flood control alternatives in reducing flood stages and corresponding damages in the Centralia-Chehalis floodplain, a baseline flood model was developed. The baseline flood model represents the existing conditions of the Upper Chehalis River Basin above the Porter gage including the recent completion of the Long Road Dike Project construction in February 2001. Development of the model was based on the February 1996 flood, which represents the new 100-year base flood in the mainstem of the Chehalis River. This flood event is the largest flood of record, and provides the most recent and complete observed flood stage data, allowing extensive calibration of the model. Upon calibration for the February 1996 flood, the model was verified against three other major flood events: the January and November 1990, and the January 1972 floods. The model developed for calibration and verification against these selected historical flood events does not include the Long Road Dike Project, slightly differing from the baseline flood model.

3.2 METHODOLOGY

The floodplain and floodway in the Centralia-Chehalis area present a complex flood hydraulic problem because of flat gradients, flow reversals, overland flow exchanges between subbasins, and local ponding created by existing dikes, levees, railroad embankments, bridge abutments, and Interstate 5 (I-5) fill in the floodplain. To adequately reproduce the historical flood flow and stage hydrographs in this area, the HEC-UNET (USACE 1996) software recently developed by Dr. Robert L. Barkau for the USACE Hydrologic Engineering Center (HEC) was used to model the upper Chehalis River Basin.

UNET is a one-dimensional, unsteady flow flood routing model that can simulate flood flow in a complex network of open channels including off-channel storage and overbank storage areas, as well as the split of flow into two or more channels and the combining of flow. The channel cross-section data used in the HEC-2 (USACE 1990) models previously developed by others (steady-state backwater model) can be readily adapted to the UNET input. Other input data includes flow and stage hydrographs, overflow spillways, bridges, culverts, and levee systems. Because of its capability to include off-channel and overbank storage areas, UNET is a quasi two-dimensional model and is considered to be the best tool available for modeling the upper Chehalis River Basin floods.

A stream network diagram of the UNET model for the Upper Chehalis River Basin, above the USGS streamgage at Porter, is provided in Figure 3-1. This figure shows the locations of 23 channel routing reaches and 69 overbank storage areas. Figure 3-2 shows how the subbasins were divided, and Table 3-1 tabulates the drainage areas for all 68 subbasins used in the Upper Chehalis River Basin UNET model. These routing reaches and subbasins were configured to facilitate accurate modeling without overly burdening the effort.

The UNET model requires input of flow hydrographs from all of the drainage subbasins at various stream locations, to account for total flood flow contribution in the upper Chehalis River stream network. Among these subbasins, three are gaged and the remaining are ungaged.

For the gaged subbasins, observed flow hydrographs were used as a direct input to the UNET model. For the ungaged subbasins, flood runoff hydrographs were simulated using USACE's HEC-1 computer program (Dodson 1995).

The HEC-1 program is a single-event flood rainfall runoff model which simulates flood runoff hydrographs from storm precipitation, taking into account antecedent ground conditions, loss rates, base flow, and snowmelt. The runoff hydrograph from each Chehalis River subbasin's response to a storm was derived by application of the Clark's unit hydrograph methodology to rainfall and snowmelt excesses.

A two-step approach was used in the HEC-1 modeling of the runoff from the upper Chehalis River subbasins. First, unit hydrograph base flow and loss rate parameters were optimized to achieve a best-fit with respect to observed hydrographs for gaged subbasins. Second, these optimized parameters were used with appropriate adjustments for drainage area and hydrologic characteristics (such as the time-of-concentration) for the rainfall runoff modeling of ungaged subbasins. Other HEC-1 input data included stream gage hydrographs, storm precipitation, and various meteorological and hydrological parameters.

Both UNET and HEC-1 use a large quantity of hydrologic data, including input and output data. The HEC-DSS program (USACE 1995) was used to provide a database system that enabled both UNET and HEC-1 to conveniently store and retrieve data from a central storage in a common format. The HEC-DSS database system used in this study includes observed hourly flow and stage hydrographs, hourly rainfall data, computed hourly flow, velocity, and stage hydrographs, and computed maximum flow, velocity and stage profiles.

Four recent major floods were selected for the Chehalis River Basin HEC-1 and UNET modeling: the February 1996, January and November 1990, and January 1972 floods. These events represent a spread of flood frequency between 15- and 100-year return intervals in the mainstem of the Chehalis River (Table 2-7). Selection of these floods for the modeling was based on criteria including availability and reliability of adequate observed meteorological and flood stage data, significant flooding in the Centralia-Chehalis area, and a representative spread of flood recurrence intervals. The computation steps for both HEC-1 and UNET were chosen to be on an hourly basis considering the drainage size and the modeling accuracy.

Further discussion of the HEC-1 and UNET model development for the upper Chehalis River Basin is provided in the following subsections.

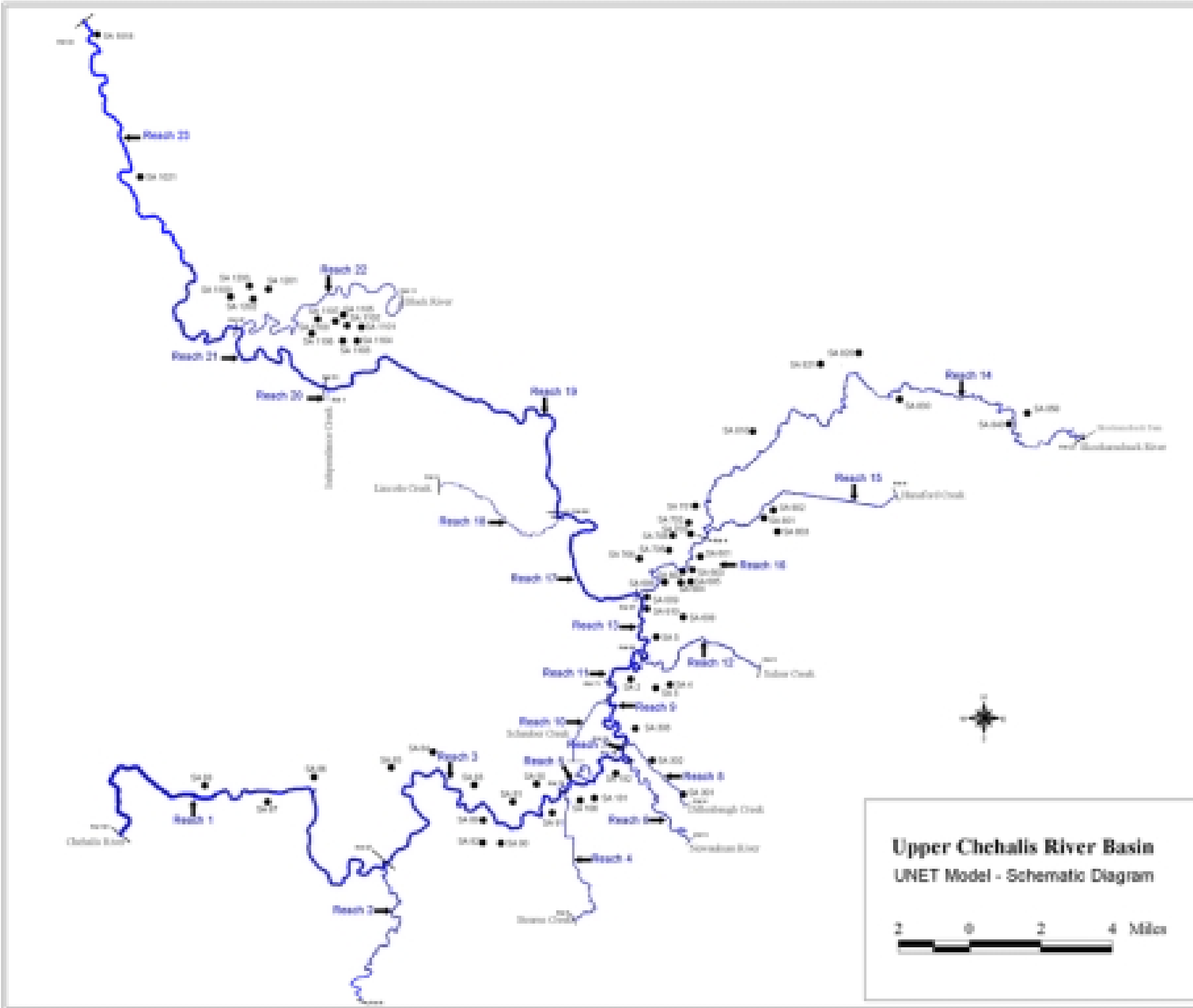


Figure 3-1: Upper Chehalis River Basin UNET Model – Schematic Diagram

Table 3-1: Upper Chehalis River Subbasin Division Summary

Symbol (see Fig. 3-2)	Subbasin Stream Name	Drainage Area (sq. mi.)	UNET Routing Reach (see Fig. 3-1)
C1	Chehalis River above Doty	113.00	1
C2	Elk Creek	46.70	1
C3	small creek d/s of Elk Creek, enters at RM 99.77	3.13	1
C4	Dunn Creek, enters at RM 98.47	5.56	1
C5	Marcusson Creek, enters at RM 97.06	2.94	1
C6	Dell Creek, enters at RM 95.16	3.63	1
C7	Garrett & Nicholson Creeks, lateral inflow to SA 86	4.00	1
C8	Hope Creek, lateral inflow to SA 87	6.30	1
C9	uniform flow area, RM 101.8-99.77	5.00	1
C10	uniform flow area, RM 99.77-94.76	13.75	1
C11	uniform flow area, RM 94.76-87.91	9.71	1
SF1	South Fork Chehalis River	52.42	2
SF2	Stillman Creek, enters at RM 5.29	45.14	2
SF3	Lake Creek, enters at RM 1.24	24.34	2
SF4	uniform flow area, RM 5.29-0.0	6.62	2
C12	Bunker Creek, lateral inflow to SA 85	34.10	3
C13	Van Orum Creek, lateral inflow to SA 84	2.16	3
C14	small creek d/s of Van Orum Creek, lateral inflow to SA 83	1.25	3
C15	small creek u/s of Stearns Creek, lateral inflow to SA 82	4.10	3
C16	Mill Creek, lateral inflow to SA 80	6.56	3
C17	uniform flow area, RM 87.90-77.96	16.98	3
ST1	Stearns Creek	23.23	4
ST2	uniform flow area, RM 4.62-0.0	11.61	4
C18	uniform flow area, RM 77.95-75.21	2.49	5
N1	Newaukum River	138.35	6
N2	uniform flow area, RM 4.11-0.1	8.08	6
C19	uniform flow area, RM 75.20-74.74	5.37	7
D1	Berwick Creek	3.51	8
D2	Dillenbaugh Creek, enters at RM 2.86	6.50	8
D3	uniform flow area, RM 3.45-0.029	2.16	8
SC1	Scheuber Creek	3.51	10
C20	uniform flow area, RM 71.48-69.23	6.43	11
SA1	Salzer Creek	12.21	12
SA2	Coal Creek	5.37	12
SA3	uniform flow area, RM 5.21-0.0	3.22	12
C21	China Creek, lateral inflow to SA 603	4.32	13
C22	Centralia area, lateral inflow to SA 609	1.00	13
S1	Skookumchuck River above dam	62.00	14
S2	Bloody Run Creek, enters at RM 21.31	3.90	14

Symbol (see Fig. 3-2)	Subbasin Stream Name	Drainage Area (sq. mi.)	UNET Routing Reach (see Fig. 3-1)
S3	Thompson Creek, lateral inflow to SA 850	7.09	14
S4	John Creek, lateral inflow to SA 840	9.77	14
S5	Salmon Creek, enters at RM 17.52	4.52	14
S6	small south side creek, lateral inflow to SA 830	3.17	14
S7	Connor Creek, later inflow to SA 810	3.29	14
S8	uniform flow area, RM 21.05-17.52	2.89	14
S9	uniform flow area, RM 17.52-11.92	10.71	14
S10	uniform flow area, RM 11.93-6.17	4.64	14
H1	Hanaford Creek	21.10	15
H2	Packwood Creek, enters at RM 5.604	7.71	15
H3	North Hanaford Creek, lateral inflow to SA 804	6.93	15
H4	South Hanaford Creek, lateral inflow to SA 803	15.00	15
H5	uniform flow area, RM 6.278-0.0	8.16	15
S11	Coffee Creek, lateral inflow to SA 704	6.21	16
S12	uniform flow area, RM 3.84-0.0	3.76	16
C23	uniform flow area, RM 66.88-61.71	10.74	17
L1	Lincoln Creek	31.84	18
L2	uniform flow area, RM 3.9-0.0	10.74	18
C24	Scatter Creek, enters at RM 54.90	41.43	19
C25	uniform flow area, RM 58.91-51.07	24.40	19
I1	Independence Creek	26.00	20
C26	uniform flow area, RM 51.06-47.42	18.30	21
B1	Black River	112.32	22
B2	uniform flow area, RM 11.11-0.0		22
C27	Garrad Creek, enters at RM 46.80	26.00	23
C28	Shelton Creek, lateral inflow to SA 1021	35.94	23
C29	Rock Creek, lateral inflow to SA 1021	24.80	23
C30	Porter Creek, lateral inflow to SA 1018	35.30	23
C31	Uniform flow area		23

3.3 SUBBASIN RAINFALL-RUNOFF MODELING

3.3.1 General

The subbasin rainfall runoff modeling through application of the HEC-1 program produced flow hydrographs required as input to the UNET flood routing model for ungaged subbasins. The HEC-1 modeling requires input of subbasin drainage geometric data, meteorological data, hydrological parameters including Clark's unit hydrograph parameters, precipitation losses, and base flow estimates. To improve the accuracy of estimating ungaged subbasin flow hydrographs, hydrological parameters were optimized using observed hydrographs at gaged subbasins. The optimized hydrological parameters were then adjusted for application to the ungaged subbasins.

3.3.2 Subbasin Definition

All subbasin geometric data, including drainage boundary, area, stream length, and slope, were delineated by utilizing the Watershed Modeling System (WMS) developed by the Engineering Graphics Laboratory of Brigham Young University (BYU) in cooperation with the USACE, Waterways Experiment Station (WES) (BYU 1996). The digital terrain modeling functions of WMS were used to create terrain models using Triangulated Irregular Networks (TINs), which automatically delineated watersheds, streams, subbasins, and all required geometric data.

3.3.3 Meteorological Input

The network of meteorological stations used in the study consisted of daily and hourly reporting climatological stations in and near the Chehalis River Basin. A total of 12 reporting precipitation stations were used. The stations and the type of data (either daily or hourly) for each station used in the HEC-1 modeling are listed in Table 2-1. Station locations are shown in Figure 2-1.

The station records available for each storm period differ due to equipment or recording problems that result in data missing for some of the stations. To help fill gaps in the hourly precipitation records, daily reporting precipitation was converted to hourly precipitation based on the nearest hourly reporting precipitation patterns. The subbasin average total and time distribution of storm precipitation were computed based on a composite weighted precipitation method. The accumulated rainfall data recorded at several hourly climatological stations for each of the four selected storm events are shown in Figure 3-3.

Figure 3-2: Upper Chehalis River Subbasin Division Map

Preliminary examination of gaged runoff for the upper Skookumchuck River Valley indicated that the adjoining meteorological stations of Centralia and Olympia did not appear to adequately account for the strong orographic rainfall component present in the upper Skookumchuck Valley. In this particular case, data from the Frances, Doty, and Cinebar stations was combined with data from the Centralia and Olympia stations in order to properly account for the orographic effects.

3.3.4 Optimization of Hydrological Parameters for Gaged Subbasins

Modeling flood runoff with the HEC-1 program requires complete definition of unit hydrograph and precipitation loss rate criteria for each subbasin within the upper Chehalis drainage area. The controlling parameters can be estimated by correlating flood runoffs with storm precipitation, using a suitable number of gaged subbasins. HEC-1 provides an optimization subroutine in which these variables are optimized by comparing the simulated flood (derived from rainfall volume) and its time distribution and drainage area, with the observed flood hydrograph. The “best” reconstitution is considered to be that which minimizes the weighted squared deviations between the observed hydrograph and a reconstituted hydrograph.

This optimization process for unit hydrograph parameters and ground loss rates was carried out for three upper Chehalis River subbasins having historical records of flood hydrographs and storm precipitation. These subbasins are the Chehalis River above Doty, Newaukum River, and Skookumchuck River above Vail.

The HEC-1 computer program derives unit hydrographs by the Clark Method. The Clark Method requires two parameters: time of concentration (*Tc*) and basin storage coefficient (*R*), both in hours. Loss rates were typically computed by the HEC exponential loss rate function, which relates loss rates to rainfall and to accumulated losses. For some of the subbasins, an initial and uniform loss rate was used. With this method, all rainfall is lost until the volume of initial loss is satisfied. After the initial loss is satisfied, rainfall is lost at a constant rate. Both the loss rate parameters and unit hydrograph parameters were determined through the process of optimization. Each of these optimizations led to a reasonably consistent, though slightly different, set of values from event to event in the same subbasin. The optimization results of unit hydrograph parameters are summarized in Table 3-2.

The base flow quantities were also estimated through the optimization process. Base flow was determined from the exponential recession limb preceding the storm runoff hydrograph. This base flow was added to the computed runoff hydrograph ordinates to obtain the total subbasin hydrograph. When the base flow is below a recession threshold flow, the program prevents it from receding faster by using the pre-flood base flow recession rate.

The reproduced and observed flow hydrographs for the selected four flood events at Doty, Newaukum, and Vail subbasins are shown in Figures 3-4 through 3-7, and indicate reasonable results of the optimization.

Table 3-2: HEC-1 Optimization Results

Subbasin/ Flood Event	Clark’s Unit Hydrograph Parameters (Hours)	
	Tc	R
Chehalis River above Doty:		
Feb-96	5.21	8.88
Nov-90	5.70	9.70
Jan-90	4.33	7.37
Jan-72	5.36	9.13
Newaukum River basin:		
Feb-96	10.45	17.80
Nov-90	12.41	21.12
Jan-90	12.30	20.95
Jan-72	12.76	21.73
Skookumchuck River above Vail:		
Feb-96	4.57	6.85
Nov-90	6.26	9.39
Jan-90	4.35	6.52
Jan-72	7.36	11.04

3.3.5 Derivation of Hydrological Parameters for Ungaged Subbasins

Upon optimization of hydrological parameters for gaged subbasins, a consistent relationship between the two Clark's unit hydrograph parameters (*R* and *Tc*) was established. A constant ratio of $R/(Tc+R) = 0.63$ was used for all subbasins and flood events.

The *Tc* parameter as optimized by HEC-1 was then compared with a computed *Tc* using the Kirpich Equation (Chow 1964), resulting in an adjustment factor being applied to the computed *Tc* value for each gaged subbasin and flood event. Applying a similar adjustment *Tc* factor and the constant $R/(Tc+R)$ ratio to a *Tc* value computed by the Kirpich Equation, final values for both *Tc* and *R* were derived and used as input to the HEC-1 rainfall runoff model for each of the 65 ungaged subbasins for each of the five selected flood events.

Other hydrological parameters, including precipitation losses and base flows for the ungaged subbasins, were estimated and were part of the HEC-1 input for flow hydrograph computations.

For small, ungaged subbasins that provide uniform flow to a UNET routing reach, hydrographs were developed with the use of index subbasins. Hydrographs for the index subbasins were first developed as described above. For uniform flow area subbasins between two index stations, hydrographs were based on a ratio of the approximate proportionate distance to the two index stations. The three index stations used were Hope Creek, Coal Creek, and Gibson Creek. Hope Creek is located just downstream of Elk Creek in the upper part of the basin and has a drainage area of 6.25 square miles. Coal Creek is roughly in the middle of the basin, flows into Salzer Creek, and has a drainage area of 5.37 square miles. Gibson Creek is located at the downstream end of the model area near Porter and has a drainage area of 5.7 square miles.

3.4 FLOOD ROUTING MODELING

3.4.1 General

As shown in Figure 3-1, the UNET model for the Upper Chehalis River Basin was developed to route flow hydrographs from headwater and intermediate subdrainage areas along the floodplain routing reaches to the downstream end at the Porter gage. Modeled flow hydrographs include observed hydrographs for gaged subbasins and computed hydrographs as described in Section 3-3 for ungaged subbasins. Stage-discharge rating data at the Porter gage provided by USGS were used as downstream boundary conditions of the UNET model.

3.4.2 Development of the UNET Model

Development of the UNET model was based on expansion of USACE's 1997 UNET model, which consists of one 21-mile reach of the Chehalis River between Adna (RM 81.14) and Grand Mound (RM 59.93). The expansion in the upper part of the basin includes a 20-mile reach above Adna to the Doty stream gage (RM 101.8), a 5.8-mile reach of the South Fork Chehalis River, and a 4.6-mile reach of Stearns Creek. In the middle portion of the basin, expansion included a 4-mile reach of the Newaukum River from its mouth to the Newaukum gage (RM 4.12), a 3.5-mile reach of Dillenbaugh Creek, a 2.6-mile reach of the Scheuber drainage ditch, a 5.2-mile reach of lower Salzer Creek, a 22-mile reach of the Skookumchuck River from its mouth to the Skookumchuck Dam (RM 21.9), and a 6.3-mile reach of Hanaford Creek. In the lower portion of the basin, expansion included a 26.6-mile reach of the Chehalis River to the Porter gage (RM 33.29), a 3.9-mile reach of Lincoln Creek, an 11-mile reach of the Black River, and a 1-mile reach of Independence Creek. The developed model includes 23 routing reaches above Porter, and 69 storage areas along the routing reaches. Characteristics of these routing reaches are provided in Table 3-3, which shows stream reach river mile range, number of cross-sections used, Manning's 'n', and contributing subbasins. The model has a total of 676 cross-sections and covers approximately 138 stream miles.

Table 3-3: Characteristics of UNET Routing Reaches

UNET Reach (see Fig. 3-1)		River MileRange (RM)	No. ofCross-sections	Range of Manning's 'n'		Contributing Subbasins (see Fig. 3-2)
No.	Stream Name			Channel	Overbank	
1	Chehalis River	101.80 to 87.91	49	0.048 to 0.055	0.07 to 0.15	C1 to C11
2	S.F. Chehalis River	5.84 to 0.00	19	0.07	0.075	SF1 to SF4
3	Chehalis River	87.90 to 77.96	42	0.045 to 0.055	0.070 to 0.12	C12 to C17
4	Stearns Creek	4.62 to 0.00	17	0.06 to 0.07	0.07	ST1 and ST2
5	Chehalis River	77.95 to 75.21	23	0.05 to 0.054	0.070 to 0.10	C18
6	Newaukum River	4.11 to 0.01	31	0.06 to 0.07	0.01 to 0.12	N1 and N2
7	Chehalis River	75.20 to 74.74	7	0.065	0.15	C19
8	Dillenbaugh Creek	3.45 to 0.0	32	0.08 to 0.09	0.15 to 0.18	D1 to D3
9	Chehalis River	74.73 to 71.49	18	0.06 to 0.065	0.09 to 0.18	N/A
10	Scheuber Drainage	2.598 to 0.0	17	0.075 to 0.08	0.12	SC1
11	Chehalis River	71.48 to 69.23	12	0.06	0.09	C20
12	Salzer Creek	5.21 to 0.02	55	0.08 to 0.09	0.075 to 0.18	SA1 to SA3
13	Chehalis River	69.22 to 66.89	38	0.042 to 0.065	0.09 to 0.18	C21 and C22
14	Skookumchuck River	21.77 to 3.85	81	0.04 to 0.08	0.10 to 0.18	S1 to S10
15	Hanaford Creek	6.278 to 0.0	44	0.07 to 0.08	0.12 to 0.18	H1 to H5
16	Skookumchuck River	3.84 to 0.0	36	0.045 to 0.08	0.12 to 0.18	S11 to S12
17	Chehalis River	66.88 to 61.71	29	0.036 to 0.052	0.10 to 0.12	C23
18	Lincoln Creek	3.9 to 0.0	14	0.07	0.15	L1 and L2
19	Chehalis River	61.70 to 51.07	25	0.038 to 0.049	0.08 to 0.13	C24 and C25
20	Independence Creek	0.95 to 0.0	7	0.065	0.12	I1
21	Chehalis River	51.06 to 47.05	20	0.038 to 0.053	0.15 to 0.20	C26
22	Black River	11.11 to 0.0	28	0.045 to 0.053	0.07 to 0.15	B1 and B2
23	Chehalis River	67.00 to 59.33	32	0.032 to 0.060	0.065 to 0.130	C27 to C31

3.4.3 Geometric Data

All cross-section data above RM 41.10 on the mainstem of the Chehalis River and outside of Thurston County, was based on 2-foot contour topographic mapping developed by the Seattle District USACE from August 1999 aerial photography. Cross-section data for areas within Thurston County was based on 2-foot contour topographic mapping developed by Thurston County from 1996 aerial photography.

Much of the bridge cross-section data for the Chehalis River reach between Grand Mound and Adna, the Newaukum River reach, and the Skookumchuck River reach, were obtained from USACE. All these data were surveyed for USACE's earlier steady-state backwater analysis during the 1970s and the 1980s. The Seattle District USACE also performed additional surveying in February and March of 2001, including data for 21 bridges within the project area. Design drawings for a number of bridges were obtained from both Lewis and Thurston County. Additional bridge and culvert design data along I-5 and SR-12 were obtained from the Washington State Department of Transportation (WSDOT).

The low flow main channel portion of the recently surveyed cross-sections used in the developed UNET model comes from the following sources:

- From RM 101.80 (Doty gage) to RM 75.09 of the Chehalis River, the Seattle District USACE surveyed 45 sections in Feb/Mar 2001.
- From RM 65.90 to RM 41.89 of the Chehalis River, the Seattle District USACE surveyed 25 sections in Feb/Mar 2001.
- From RM 5.84 to RM 0.14 of the South Fork Chehalis River, the Seattle District USACE surveyed seven sections in Feb/Mar 2001.
- From RM 3.80 (downstream of the Newaukum gage) to RM 1.30 of the Newaukum River, the Seattle District USACE surveyed three sections in Feb/Mar 2001.
- From RM 21.31 to RM 4.80 of the Skookumchuck River, the Seattle District USACE surveyed 15 sections in Feb/Mar 2001.
- From RM 11.11 to RM 0.20 of the Black River, the Seattle District USACE surveyed 15 sections in Feb/Mar 2001.
- 12 new cross-sections were surveyed in 1997 by PI Engineering's survey subconsultant, Duane Hartman and Associates, Inc. (DHA) within the 3-mile "hump" reach of the Chehalis River below the Skookumchuck River mouth.
- From RM 67 to RM 76, including the lower Newaukum River to RM 1.49, DHA surveyed 35 sections in 1998.

For model cross-sections without recent channel survey data, the low flow channel was estimated based on nearby surveyed sections, as well as surveyed sections from USACE's steady-state backwater analysis from the 1970s and 1980s. Specific cross-section source data is noted in each cross-section of the final UNET model.

The upstream boundary of the model in the Newaukum River reach is the Labree Road Bridge at RM 4.11. Upstream of the bridge, high flows break out across the north bank of the Newaukum River and flow first into Berwick Creek, then into Dillenbaugh Creek. To account for this flow split, a separate reach was created to model the flows entering Dillenbaugh Creek. A preliminary UNET model, extending up the Newaukum River above the Labree Road overflow area at the right bank, was used to estimate the 100-year flood hydrograph for flows overflowing into Dillenbaugh Creek upstream of the Labree Road Bridge. Preliminary modeling indicated that for the floods examined, only the 100-year or larger floods had significant overflow into Dillenbaugh Creek. The January 1990 flood (70-year recurrence interval) was shown to have only negligible overflow.

3.4.4 UNET Model Calibration

The Upper Chehalis River Basin UNET model was initially calibrated using observed stage and flow hydrographs at the Mellen Street, Pearl Street, Bucoda, and Grand Mound gages for the February 1996 flood event. The calibration procedures primarily involved adjusting both channel and overbank Manning's 'n' values, as well as the geometry (both elevation and width) of storage areas and overflow connections. Upon satisfactory calibration of the stage and flow hydrographs, further calibration was performed using high water mark data provided by USACE, the City of Centralia, the City of Chehalis, and WSDOT. The original calibration model (referred to as the May 15, 2001 model) was submitted to USACE to be reviewed by WEST Consultants. The UNET model was modified to incorporate the review comments. The updated model is referred to as the September 20, 2001 model. The calibration results of the September 20, 2001 model are presented in Table 3-4, which shows a good match between the observed and the computed stage. Comparisons of stage and flow hydrographs at the Mellen Street, Pearl Street, Bucoda, and Grand Mound gages are shown in Figures 3-8 through 3-11.

Table 3-4: Comparison of Computed and Observed Maximal Water Surface Elevations (February 1996 Flood)

Stream	Location (River Mile)	Computed Elevation (ft)	Observed Elevation (ft)	Difference (ft)
Chehalis River	97.00	284.34	284.46	-0.12
Chehalis River	89.86	228.95	228.90	0.05
Chehalis River	81.03	195.64	195.97	-0.33
Chehalis River	76.19	182.73	182.53	0.20
Chehalis River	75.09	182.04	182.35	-0.31
Chehalis River	74.82	181.54	181.50	0.04
Chehalis River	74.02	179.61	179.98	-0.37
Chehalis River	72.88	178.65	178.56	0.09
Chehalis River	72.80	178.57	178.50	0.07
Chehalis River	67.86	175.83	176.21	-0.38
Chehalis River	67.44	174.04	174.30	-0.26
Chehalis River	66.88	173.01	173.14	-0.13
Chehalis River	66.75	172.64	172.21	0.43
Chehalis River	66.36	169.38	169.72	-0.34
Chehalis River	64.22	161.18	161.13	0.05

Stream	Location (River Mile)	Computed Elevation (ft)	Observed Elevation (ft)	Difference (ft)
Chehalis River	63.20	155.65	155.50	0.15
Chehalis River	62.01	153.01	153.33	-0.32
Chehalis River	59.88	143.63	143.55	0.08
Chehalis River	54.60	120.98	121.03	-0.05
Chehalis River	54.09	116.33	116.71	-0.38
Chehalis River	53.93	114.58	114.46	0.12
Chehalis River	53.90	114.41	114.45	-0.04
Chehalis River	53.30	111.42	111.00	0.42
Chehalis River	51.06	103.28	102.96	0.32
Chehalis River	49.95	96.16	96.32	-0.16
Chehalis River	45.25	83.86	83.81	0.05
Chehalis River	41.30	68.29	67.80	0.49
Chehalis River	33.29	48.95	48.86	0.09
Dillenbaugh Creek	1.25	183.56	183.70	-0.14
Dillenbaugh Creek	0.09	182.02	182.01	0.01
Salzer Creek	1.56	176.79	177.00	-0.21
Salzer Creek	1.28	176.78	177.00	-0.22
Salzer Creek	0.36	176.68	176.72	-0.04
Newaukum River	4.11	201.96	202.28	-0.32
Newaukum River	1.66	184.11	184.50	-0.39
Skookumchuck River	20.70	330.40	330.58	-0.18
Skookumchuck River	6.40	212.91	212.47	0.44
Skookumchuck River	3.84	197.98	198.26	-0.28
Skookumchuck River	2.42	187.36	187.29	0.07
Skookumchuck River	2.18	185.20	185.00	0.20
Skookumchuck River	2.00	184.39	184.30	0.09
Skookumchuck River	1.90	183.85	184.10	-0.25
Black River	9.09	109.30	109.60	-0.30
Black River	4.54	97.12	97.55	-0.43
Black River	3.45	94.21	94.08	0.13
Black River	2.48	92.25	92.72	-0.47

3.4.5 UNET Model Verification

The Upper Chehalis River Basin UNET model calibrated for the February 1996 flood event (September 20, 2001 model) was verified against observed stage and flow hydrographs at the Mellen Street, Pearl Street, Bucoda, and Grand Mound gages for the other three selected flood events, the November and January 1990, and January 1972 floods. During the model verification, Manning’s ‘n’ was at times modified slightly. Slight changes in Manning’s ‘n’ values within a reasonable range helps to account for differences between flood events due to factors such as: seasonality differences, changes in vegetative growth levels and patterns in different years and months, as well as differences in flow depths. These slight changes to Manning’s ‘n’ were back checked by running the 1996 flood event and comparing the results to the results from the calibration model. The maximum change in flow was a 3.5 percent increase at RM 70.67 for the

Manning's 'n' values used from the January 1972 flood verification. The maximum change in stage was –1.05 feet at RM 67.44, using the Manning's 'n' values from the January 1972 flood verification.

Table 3-5 shows a comparison of the computed and observed maximum water surface elevations for the model verification run for the January 1990 flood event. No high water marks were collected for the November 1990 and January 1972 flood events. Figures 3-8 through 3-23 show a comparison of the computed and observed hydrographs at the four selected gages. The comparison shows that the UNET model produces satisfactory results in reproducing these flood stage hydrographs.

The calibrated UNET model was also run for the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year statistical flood hydrographs developed by USACE. The UNET results appear to produce a flow-versus-stage rating curve at Grand Mound consistent with the USGS rating curve for that gage. Figure 3-24 shows a comparison of the USGS discharge-rating curve and the UNET computed discharge-rating curve using USACE's statistical flood hydrographs.

Table 3-5: Comparison of Computed and Observed Maximal Water Surface Elevations (January 1990 Flood)

Stream	Location (River Mile)	Computed Elevation (ft)	Observed Elevation (ft)	Difference (ft)
Chehalis River	75.09	181.55	181.54	0.01
Chehalis River	74.71	180.41	180.10	0.31
Chehalis River	72.80	178.00	177.15	0.85
Chehalis River	67.44	173.64	73.50	0.14
Chehalis River	66.30	168.47	168.74	-0.27
Chehalis River	63.20	155.19	156.93	-1.74*
Chehalis River	59.89	143.04	143.00	0.04
Chehalis River	54.09	115.91	112.66	3.25**
Chehalis River	53.93	114.40	113.56	0.84
Chehalis River	51.06	103.02	103.10	-0.08
Chehalis River	50.00	96.00	95.32	0.68
Salzer Creek	0.38	175.82	174.88	0.94
Skookumchuck River	6.40	210.80	212.30	-1.50
Skookumchuck River	4.53	200.68	204.00	-3.32^
Skookumchuck River	3.42	196.00	196.90	-0.09
Skookumchuck River	2.90	191.60	191.60	0.00
Skookumchuck River	2.42	186.95	187.10	-0.15
Skookumchuck River	1.58	179.80	180.80	-1.00
Skookumchuck River	1.52	179.00	180.00	-1.0
Skookumchuck River	0.61	174.62	174.50	0.12
Skookumchuck River	0.21	172.05	172.10	-0.05
Black River	9.00	107.60	108.45	-0.85
Black River	8.41	105.70	105.88	-0.18
Black River	7.03	102.62	101.50	1.12
Black River	6.80	100.60	100.01	0.59
Black River	6.41	98.41	99.20	-0.79
Black River	4.36	95.77	95.88	-0.11

Stream	Location (River Mile)	Computed Elevation (ft)	Observed Elevation (ft)	Difference (ft)
Black River	3.45	93.48	93.08	0.40
Black River	2.49	91.44	92.17	-0.73
Black River	1.18	89.49	90.57	-1.08
SA 202	N/A	90.13	88.23	1.90
SA 103	N/A	105.42	103.10	2.32
SA 102	N/A	99.64	99.45	0.19
SA 101	N/A	106.69	108.51	-1.82

3.4.6 UNET Model Sensitivity Analyses

A sensitivity analysis on model parameters was performed to better understand the model response and calibration. All the sensitivity analyses were based on the May 15, 2001 model except the sensitivity analysis on the parameter of Manning's 'n', which was based on September 20, 2001 model.

Implicit weighting factor THETA

The implicit weighting factor THETA was changed to 0.6 from 1.0 using the upstream model (RM 101.8-RM 59.89). A value of 1.0 provides maximum stability for the model. A value of 0.6 provides maximum accuracy, however, the model may be susceptible to instabilities. Comparing the results (2-96 flood event) with the theta value of 1.0 used in the calibration model, the maximum difference in peak flow was within 2.9 percent (RM 66.47), and maximum stage within +0.25 feet (RM 68) and –0.08 feet (RM 88).

Computational time step

A computational time step of 5 minutes was used in the calibration model (May, 15 2001 model). Time-steps of 5-minutes, and 6-minutes were run with the Feb. 1996 flood event. The difference was within a range of –0.02 feet (RM 71) to +0.04 feet (RM 87.9) for maximum water surface elevation. The maximum difference in peak flow was within 0.45 percent (RM 71.38).

Distance step XMINC

Noting the large reach length between some sections in Reach 19, a sensitivity analysis on the distance step (XMINC value in field 6 of the XK record) values was first performed in Reach 19. A smaller XMINC, approximately half of the original value, was used for Reach 19. For the Feb. 1996 flood event, the maximum stage difference at RM 59.64 was less than 0.30 feet. Above RM 59.64 and below RM 57.56, there was no significant effect.

The sensitivity analysis was then expanded to cover the entire model. To analyze the sensitivity of maximum interval in miles between interpolated cross sections for the whole model, two different Xminc values, 0.07 and 0.15, were used. The results were compared with the results in which the original value of 0.10 was used. The difference was found to be local and insignificant. The peak flow difference is within +3,038 cfs(RM 63.5) and –3,734 cfs (RM 76.36), while peak stage is within +0.33 ft (RM 54.09) to –0.4 ft (RM 54.09). In the project area, the peak stage differences are significantly less. The results are shown in Figure 3-25

Initial flow condition

All inflow hydrographs were extended (constant flow) further backward in time to the beginning of February 1, 1996, which is five days before the beginning of the Feb-96 flood event. The model was rerun without the "hot start" file involved. The comparison of the results shows that there is a slight decrease in peak flow and stage. The peak flow decreases 1 percent at Grand Mound and 0.75 percent at the Mellen Street Bridge. The maximum stage decreased 0.05 ft at Grand Mound and 0.09 ft at the Mellen Street Bridge.

Manning's 'n', storage volume and weir flow 'c' coefficients

An analysis was performed to check the sensitivity of the model to storage volumes in the Centralia-Chehalis area (RM 65.2-RM 74.02) using the calibration model. For a 50 percent increase / decrease in storage volume, the stage change for the Feb. 1996 flood event was in the range of -0.02 feet (RM 67.45) to 0.27 feet (RM 63.80). When a weir flow 'c' 2.9 was used instead of a value of 2.6, the stage change for the Feb. 1996 flood event was also in the range of -0.02 feet (RM 67.46) to 0.27 feet (RM 63.80).

An analysis was also run to check the sensitivity of the model to Manning's 'n' values in the Centralia-Chehalis area using the September 20, 2001 model. For a 20 percent increase / decrease in Manning's 'n' value, the stage change for the Feb. 1996 flood event was in the range of -1.62 ft (RM 66.36) to +1.35 ft (RM 66.36) in the Centralia-Chehalis area (RM 65.2-RM 74.02).

Changes Made to the Calibration Model

Sensitivity analyses were performed on all the review comments, but not all of the recommended changes were incorporated into the September 20, 2001 UNET model since some of the changes have only local or insignificant effects. The changes that were made to the initial May 15, 2001 model to produce the September 20, 2001 model are listed below:

1. Additional cross-sections were added for bridges at RM 100.43, RM 82.6, RM 81.0, RM 77.94 and RM 77.64 of the Chehalis River, and the bridge at RM 18.31 of the Skookumchuck River.

2. The ineffective flow area option was added to cross-sections 2 and 3 for most all of the bridges using the special bridge method of computation. The exceptions are the bridges at RM 77.94 of the Chehalis River, at RM 0.22 of the Skookumchuck River, and at RM 7.04 of the Black River.

3. The width of the effective flow areas described on the X3 records was adjusted for the bridges at RM 97.87 and RM 75.08 of the Chehalis River.

4. The X3 record describes left and right encroachment stations and elevations in fields 4 through 7. In Reach 1 at RM 97.89, the X3 record has values in fields 3 through 6. This was a mistake that has been corrected in the model.

5. The bridges at RM 18.31, RM 17.51 and RM 14.56 of the Skookumchuck River were changed from "normal bridge" method to the "special bridge" method of computation. The BT and GR cards of these bridges were also adjusted slightly according to USACE survey data of February 2001.

6. All bridges have the NC card with field 6 added.

7. The starting time in the gn.bc file was corrected to 06 Feb 1996 0500. The computation time step interval was changed from 10 minutes to 5 minutes.

8. The bridge at RM 4.685 of Salzer Creek was removed to ensure the stability of the model.

9. The bridge deck at RM 3.002 of Hanaford Creek was raised above the water to ensure the stability of the model.

10. The elevation increment in field 5 of the XK cards was changed from 3.00 to 2.75 in reach 14 and 15 to be consistent with the other reaches.

Cumulative Effects

To better understand the sensitivity of the model and the cumulative effects of all the changes, the model was modified to incorporate all the review comments. In addition to the changes made to model listed above, the following additional changes were made to the model:

1. Two new cross sections (cross-section 2 and 5) were added upstream and downstream of the BNSF Railroad Bridge at RM 9.81 of the Skookumchuck River.

2. A new cross-section was added at RM 17.80 of the Skookumchuck River, which is half way between two bridges: the bridge at RM 18.31 and the bridge at RM 17.51

3. An X3 card assigned with an ineffective flow area was added to the bridge at RM 7.04 of the Black River for the right bank.

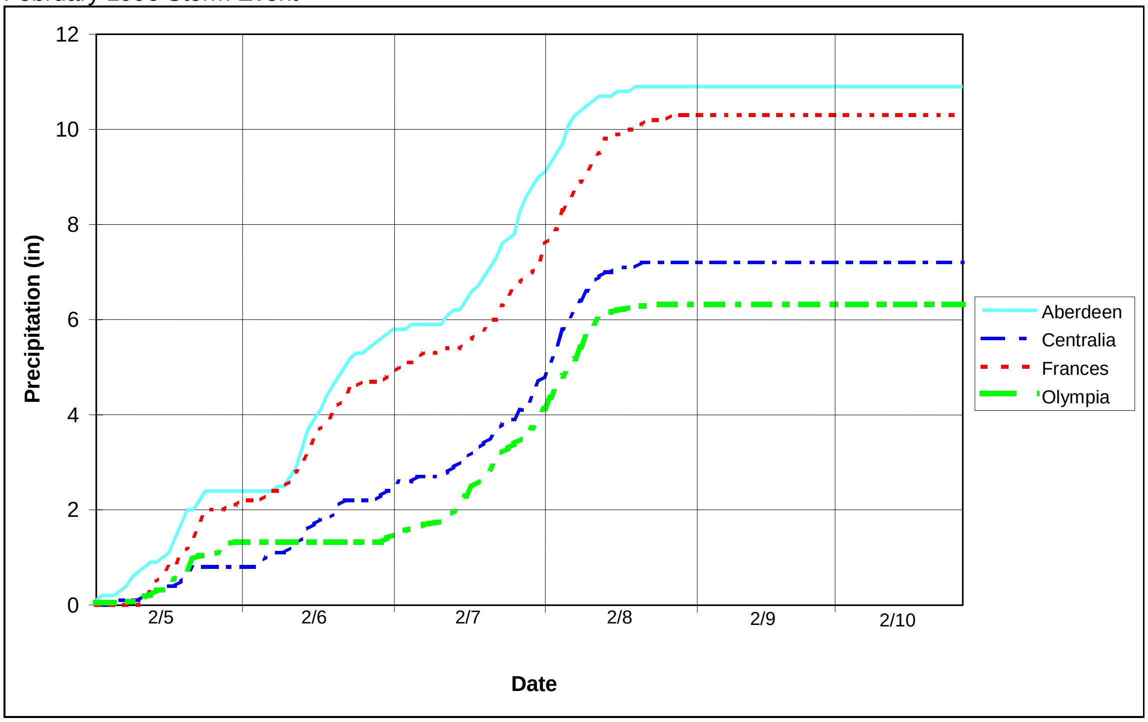
4. At bridges at RM 7.31, RM 9.81 and RM 10.85 of Skookumchuck River, the X3 cards were changed according to the “FOLLOW-UP TO BACKCHECK 1.7”(WEST Review, September 25,2001).

5. The bridge deck at RM 0.62 of Skookumchuck River was raised above the water to ensure the stability of the model.

The cumulative effects of all the changes are insignificant. Comparing with the results of May 15, 2001 model, the maximum change in stage is less than 0.5 feet at high water calibration points listed in Table 3-4. The accuracy of the computed water surface elevation is within 0.5 feet compared with observed high water marks.

Figure 3-3: Accumulated Rainfall Curves

February 1996 Storm Event



November 1990 Storm Event

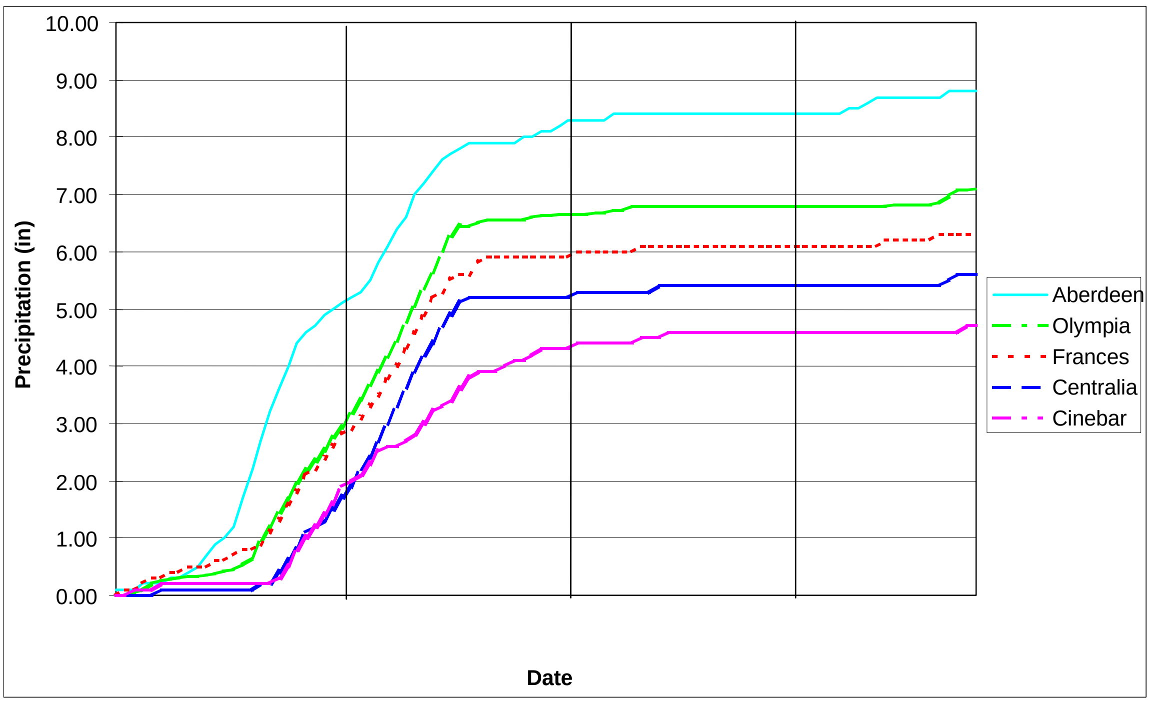
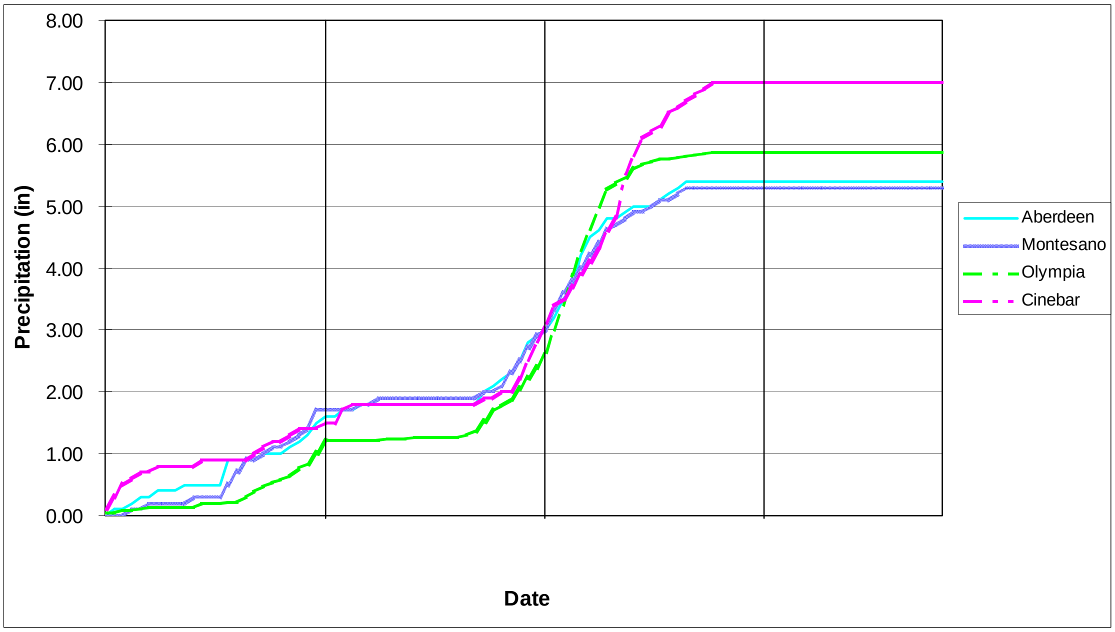


Figure 3-3:(continued)

Accumulated Rainfall Curves

January 1990 Storm Event



January 1972 Storm Event

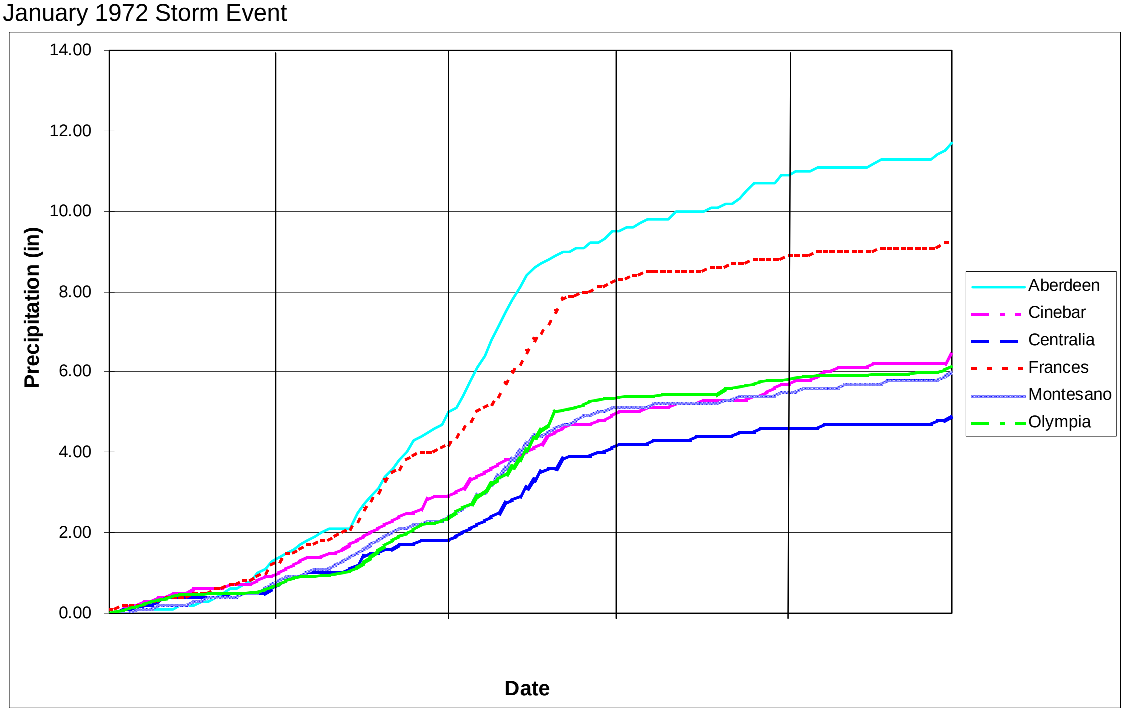
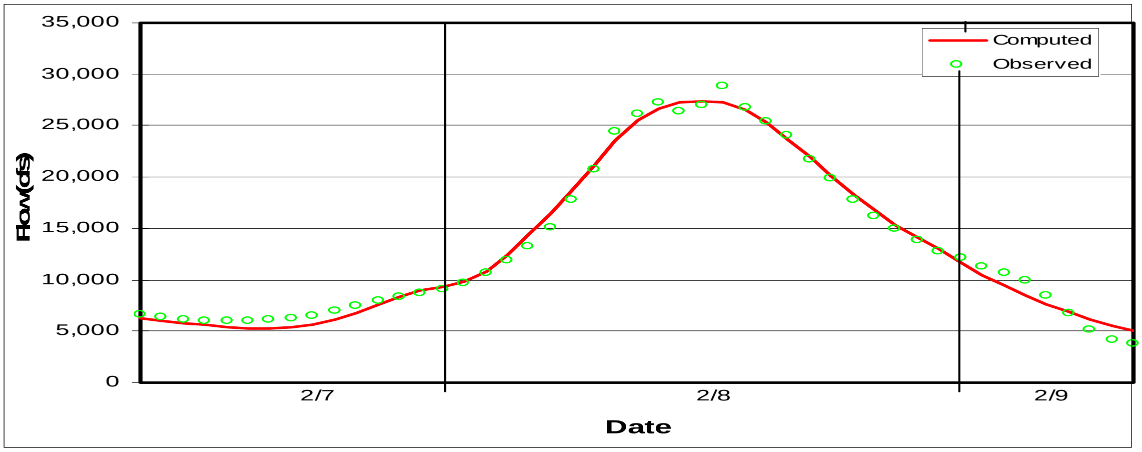
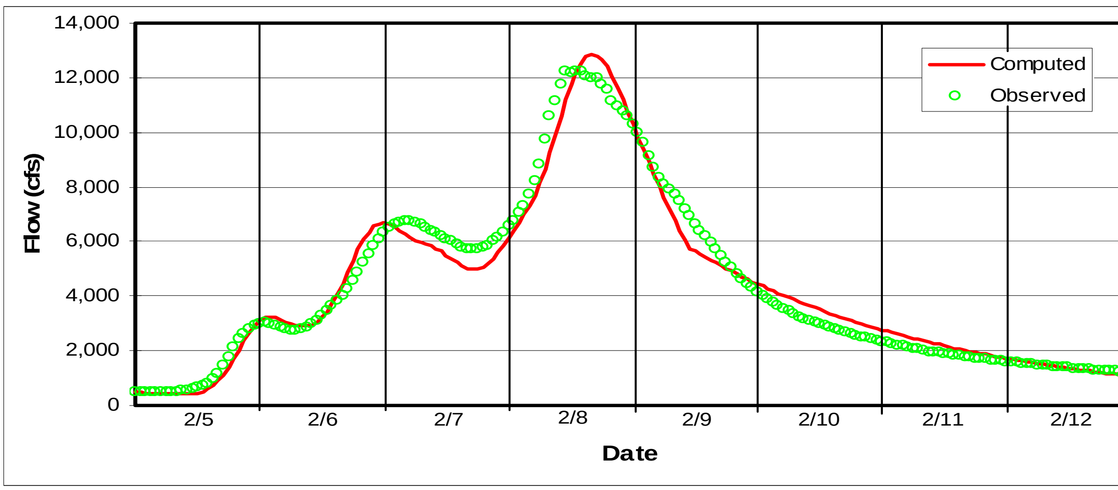


Figure 3-4: Comparison of Computed and Observed Hydrographs for the February 1996 Flood

Chehalis River near Doty



Newaukum River near Chehalis



Skookumchuck River near Vail

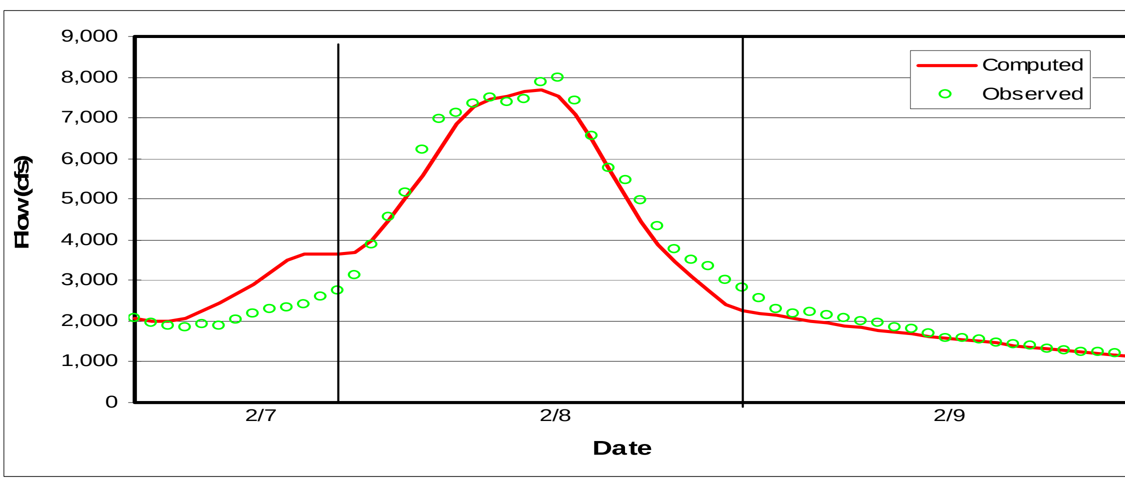
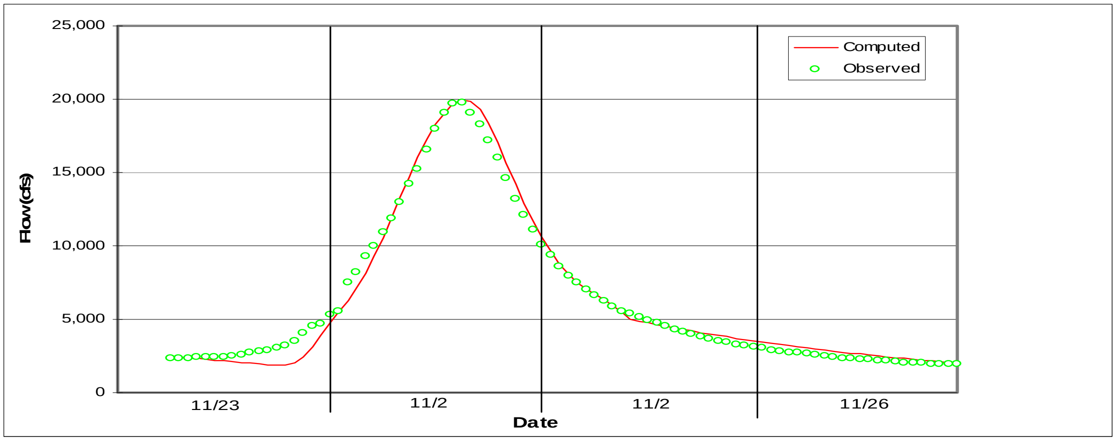
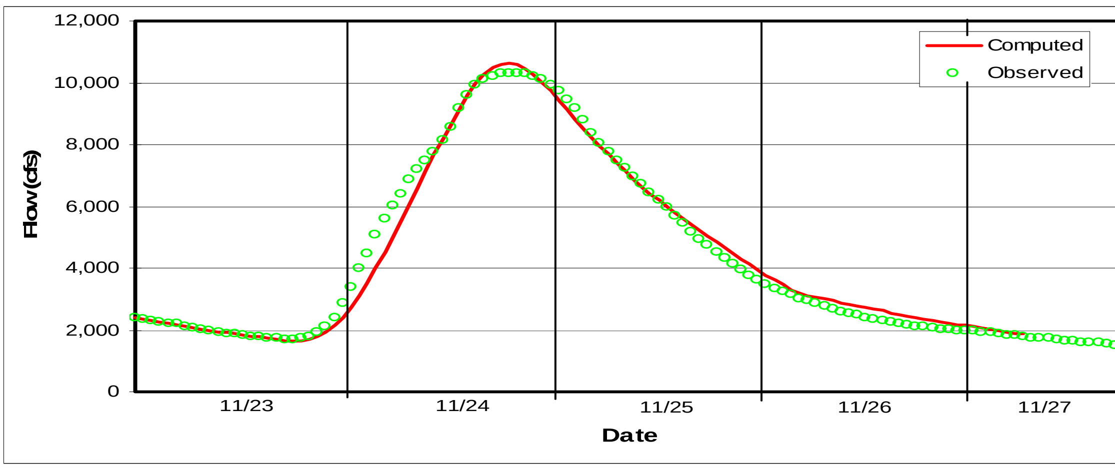


Figure 3-5: Comparison of Computed and Observed Hydrographs for the November 1990 Flood

Chehalis River near Doty



Newaukum River near Chehalis



Skookumchuck River near Vail

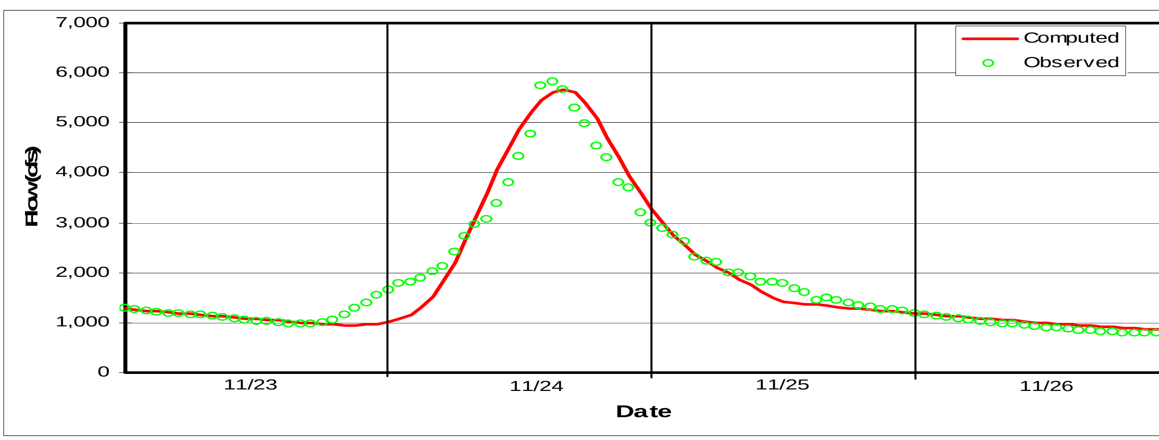
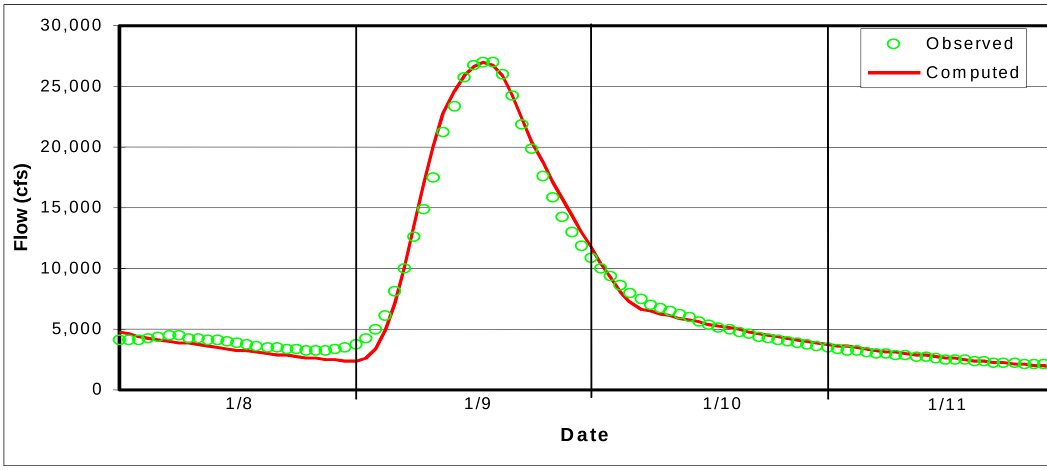
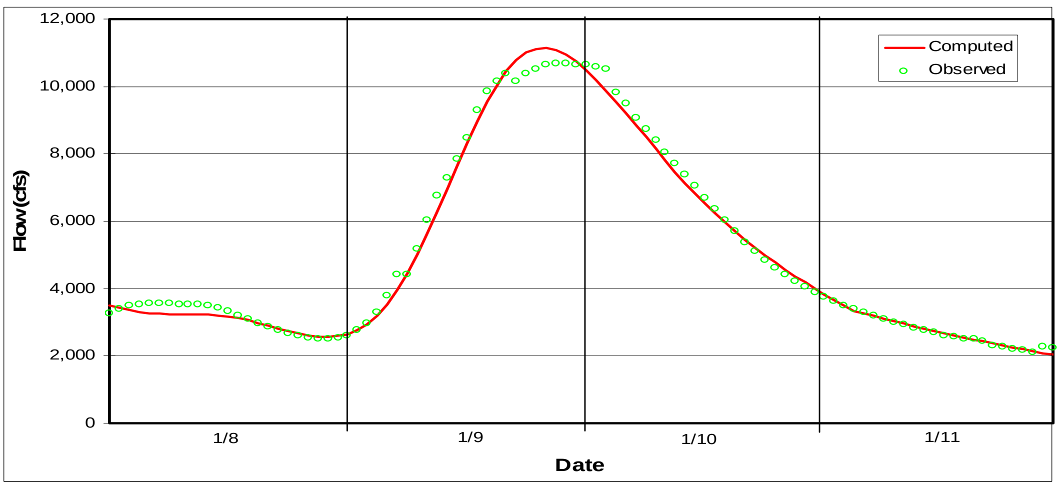


Figure 3-6: Comparison of Computed and Observed Hydrographs for the January 1990 Flood

Chehalis River near Doty



Newaukum River near Chehalis



Skookumchuck River near Vail

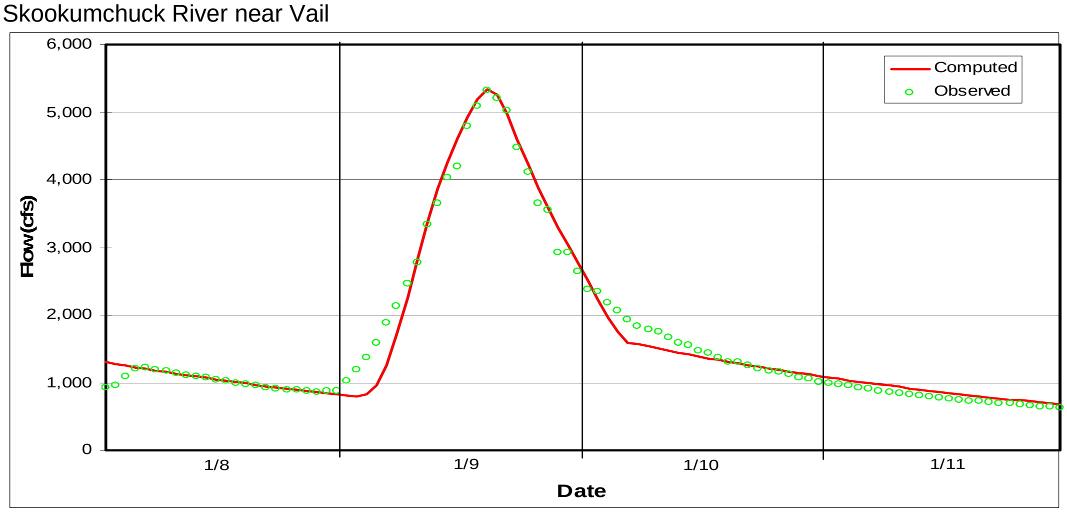
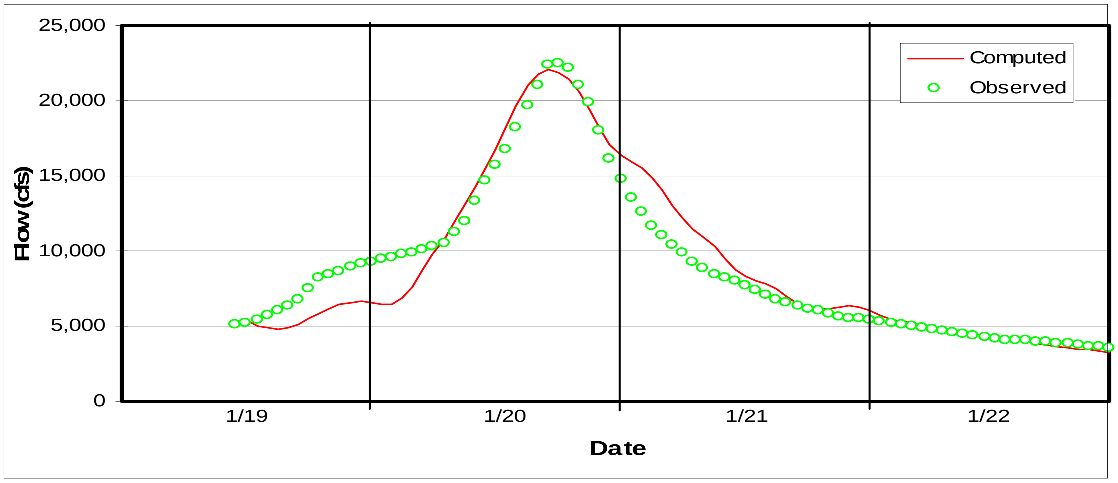
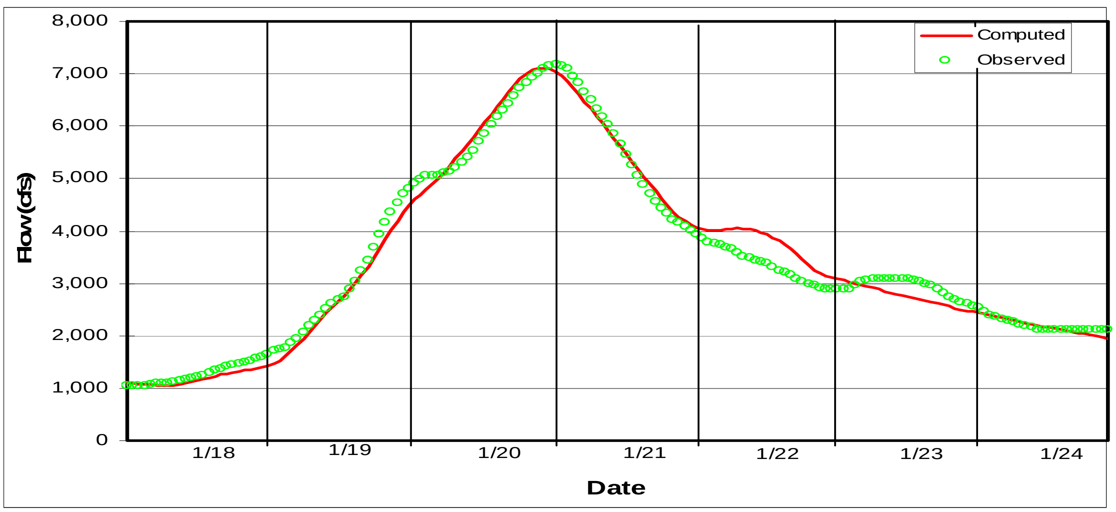


Figure 3-7: Comparison of Computed and Observed Hydrographs for the January 1972 Flood

Chehalis River near Doty



Newaukum River near Chehalis



Skookumchuck River near Vail

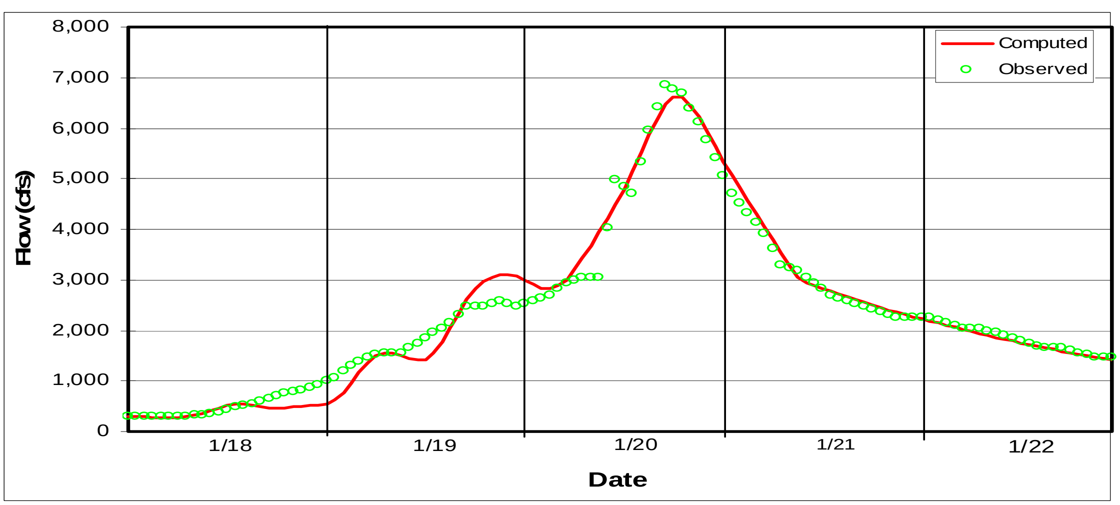
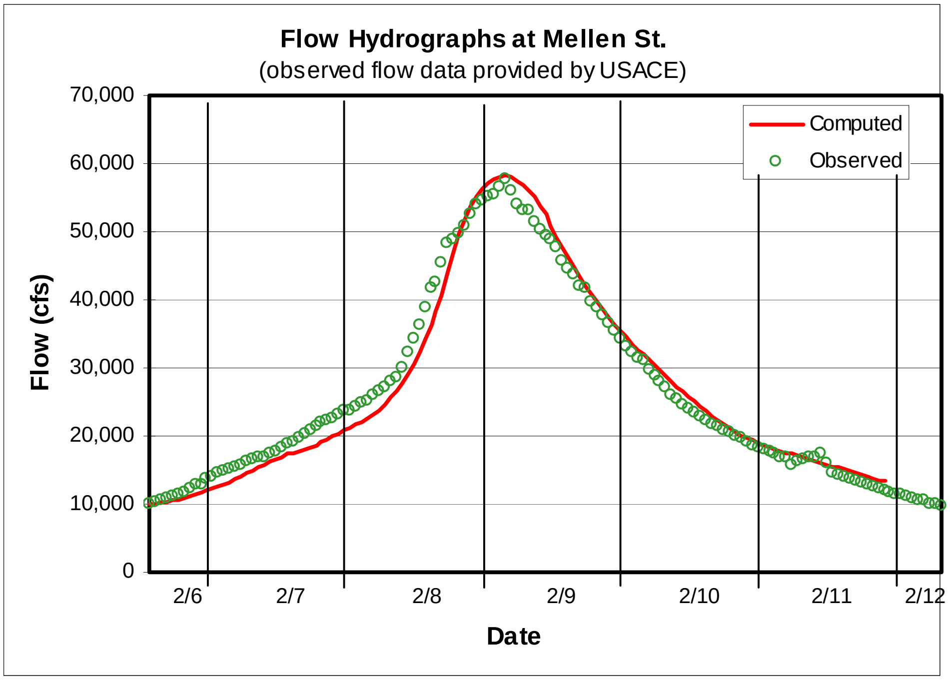


Figure 3-8: Comparison of Computed and Observed Hydrographs on the Chehalis River at Mellen Street - February 1996 Flood



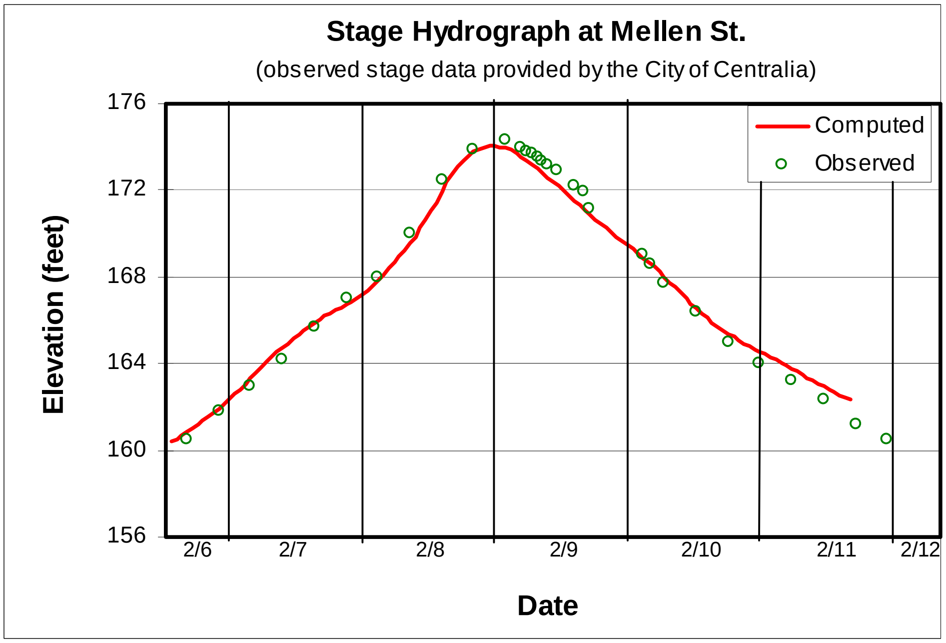
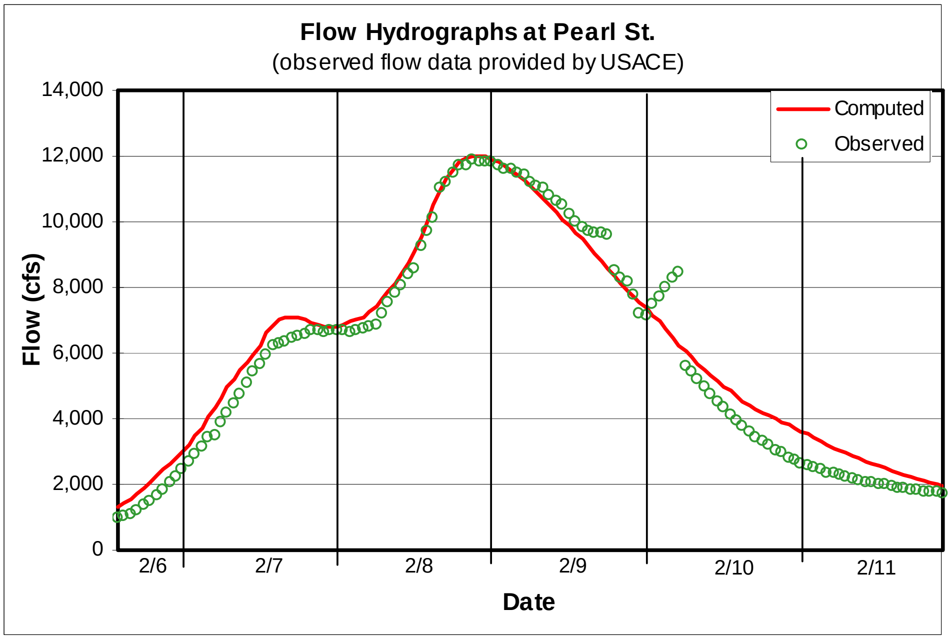


Figure 3-9: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Pearl Street - February 1996 Flood



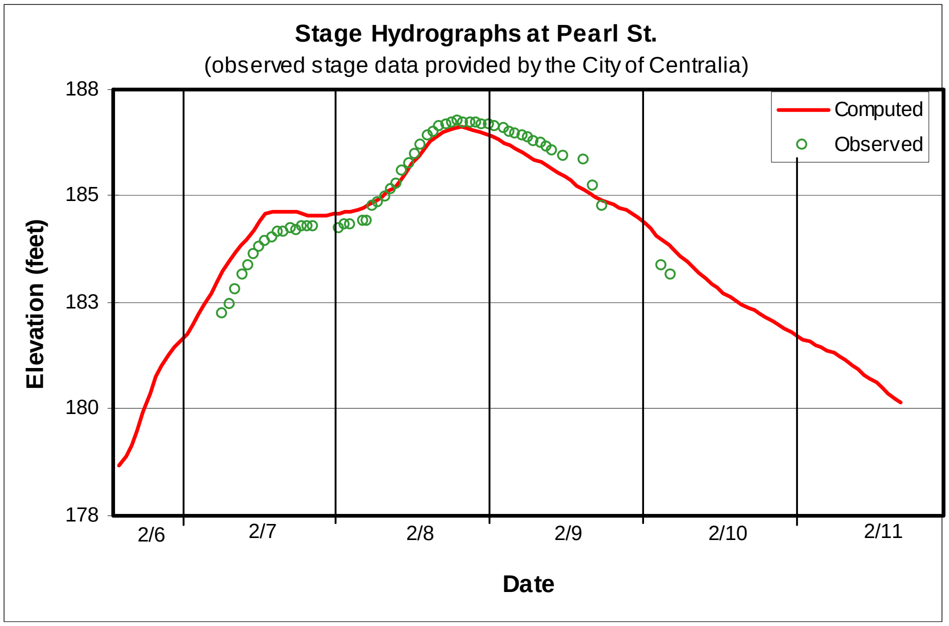
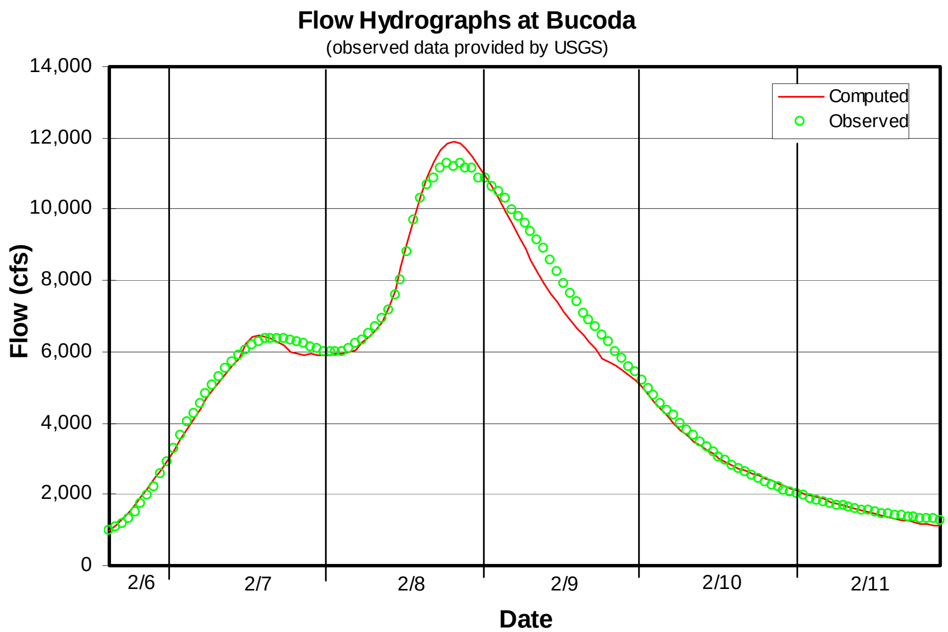


Figure 3-10: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Bucoda - February 1996 Flood



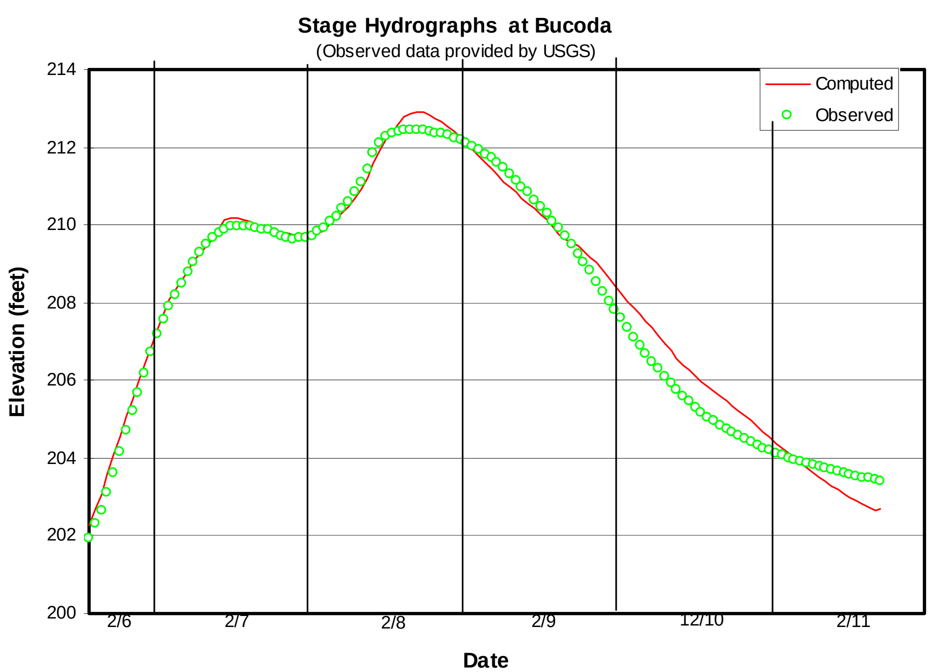
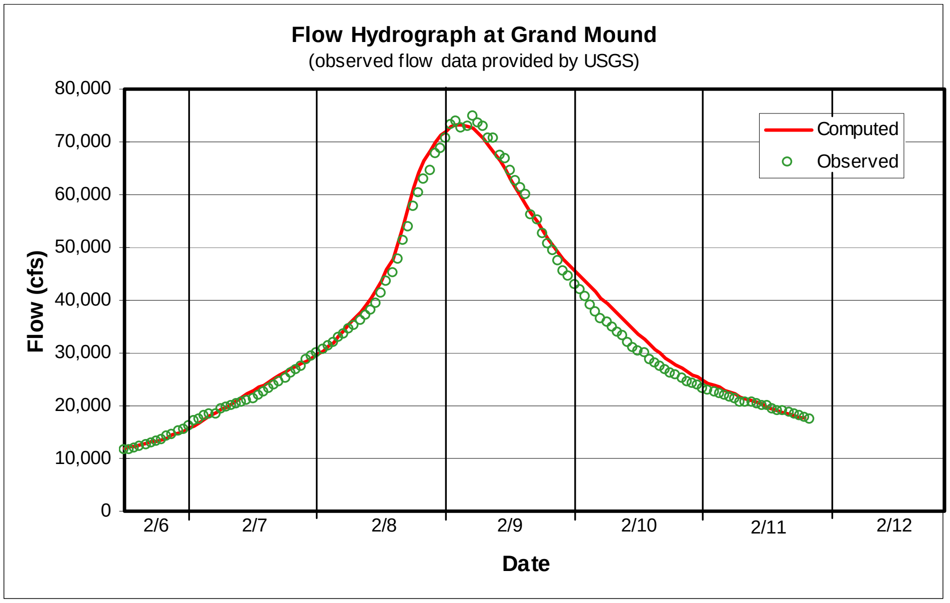


Figure 3-11: Comparison of Computed and Observed Hydrographs on the Chehalis River at Grand Mound - February 1996 Flood



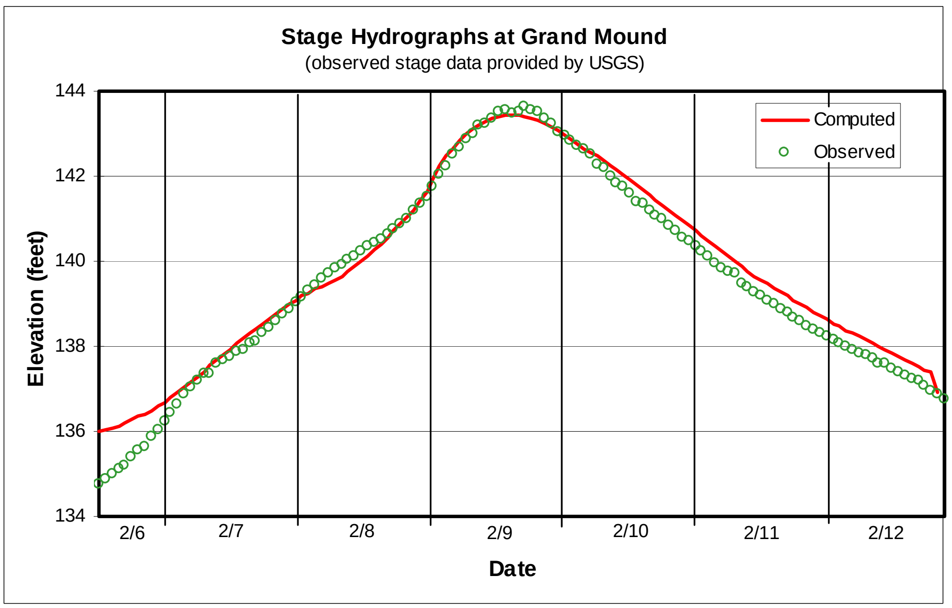
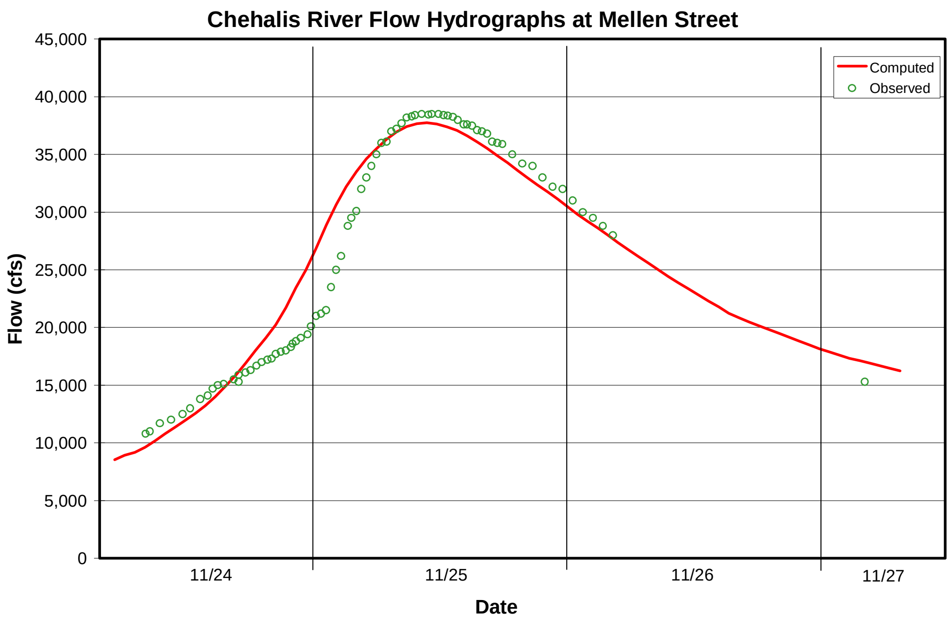


Figure 3-12: Comparison of Computed and Observed Hydrographs on the Chehalis River at Mellen Street – November 1990 Flood



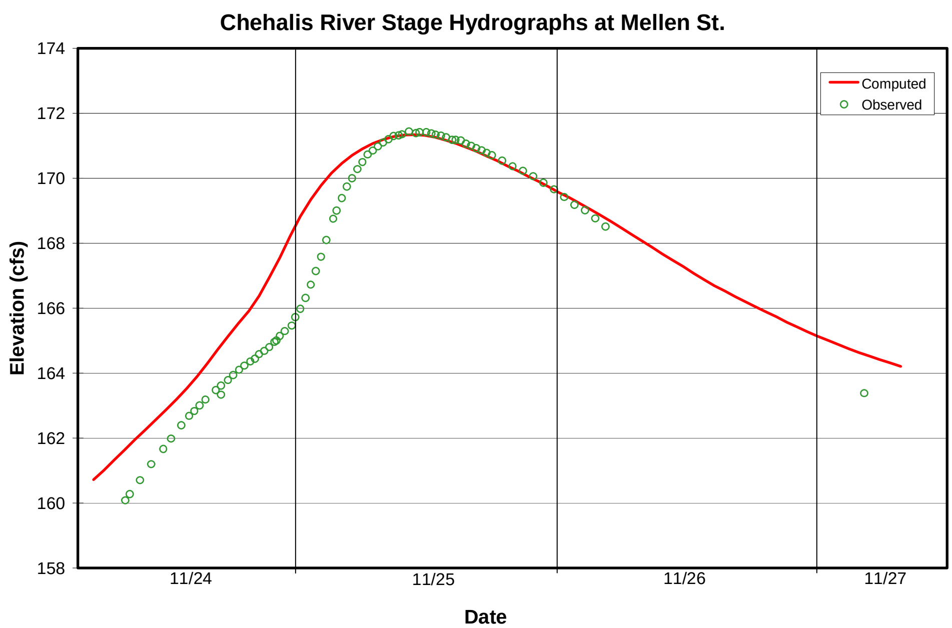
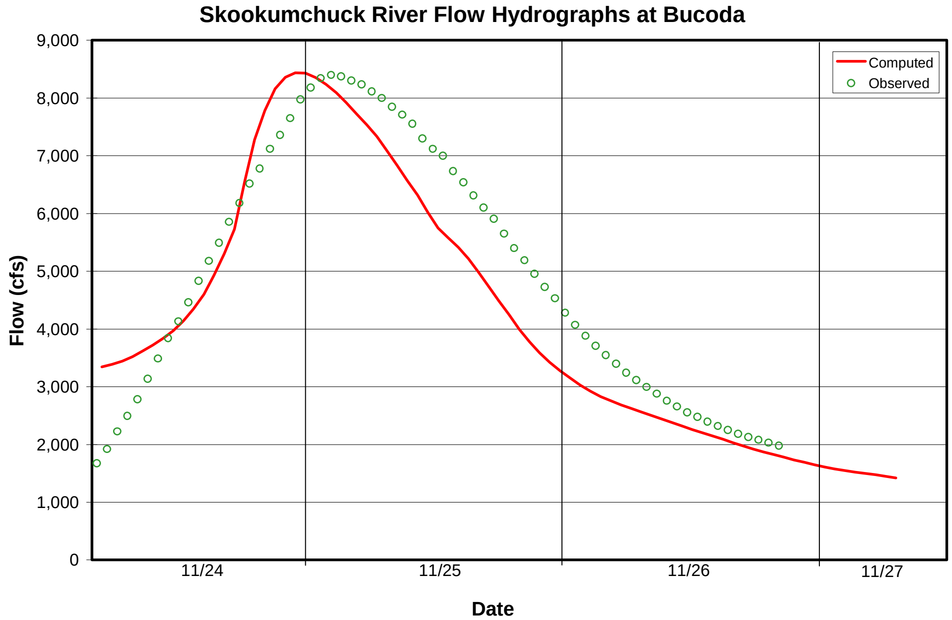


Figure 3-13: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Bucoda – November 1990 Flood



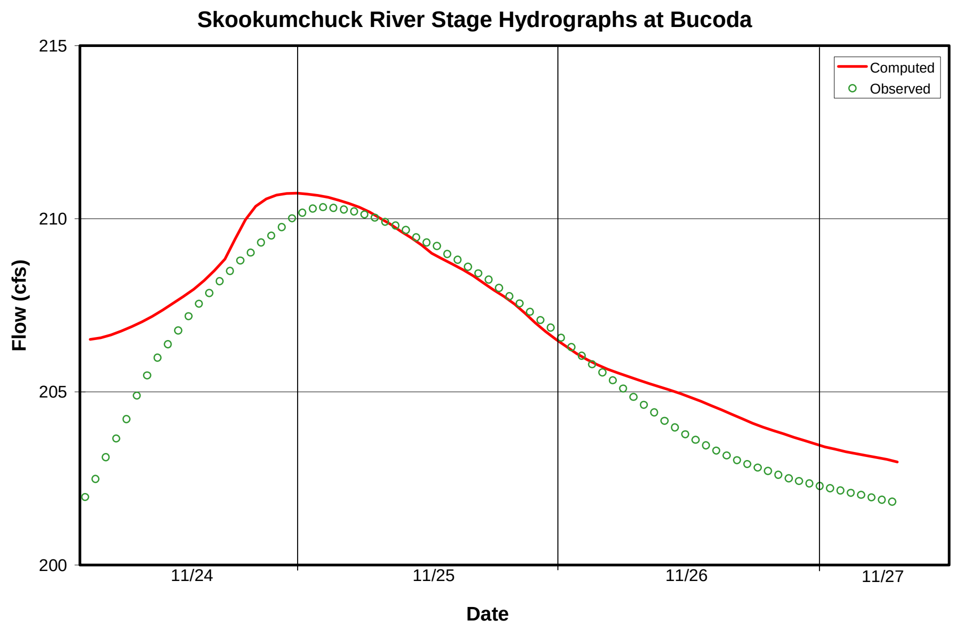


Figure 3-14: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Pearl Street – November 1990 Flood

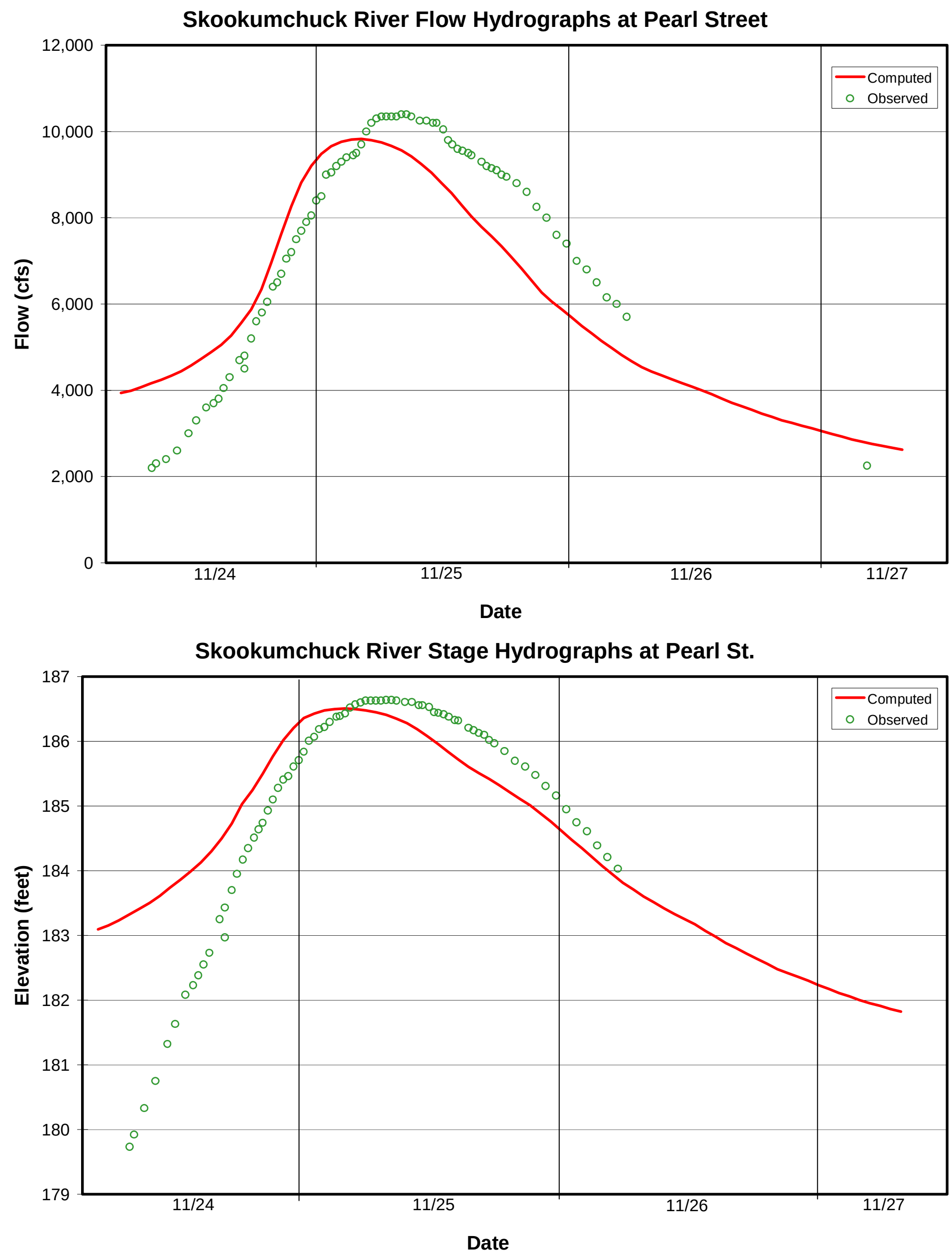
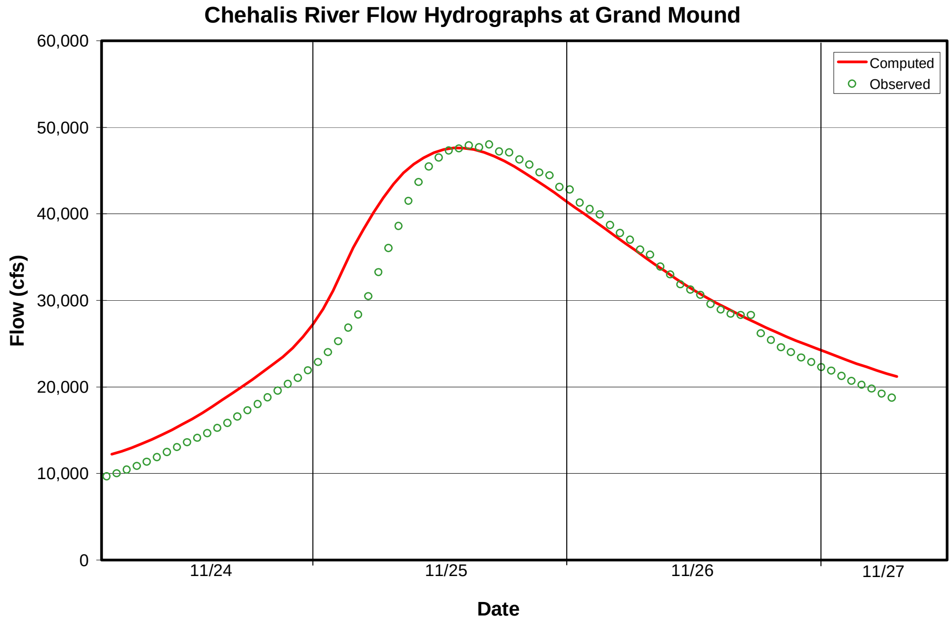


Figure 3-15: Comparison of Computed and Observed Hydrographs on the Chehalis River at Grand Mound – November 1990 Flood



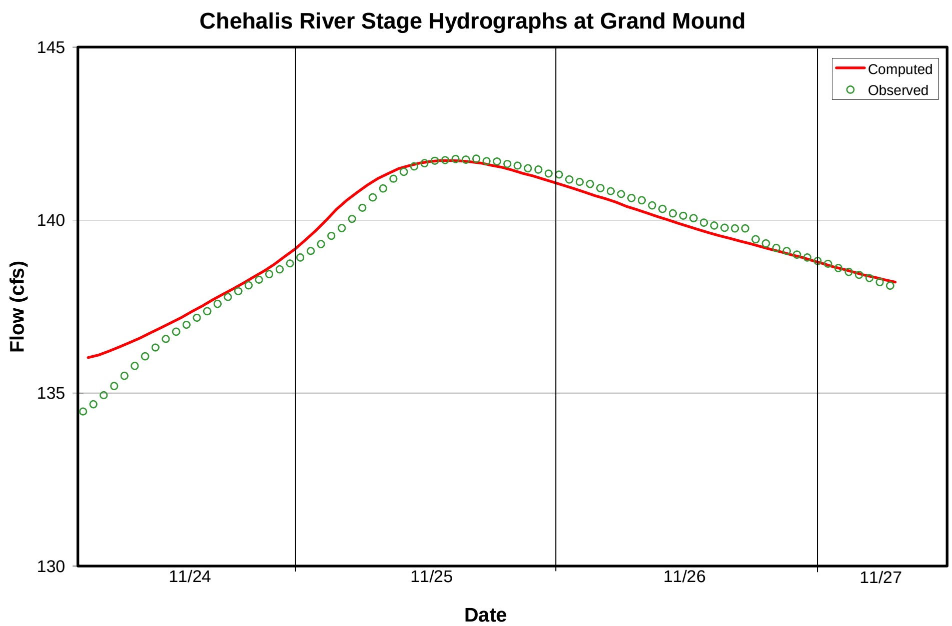
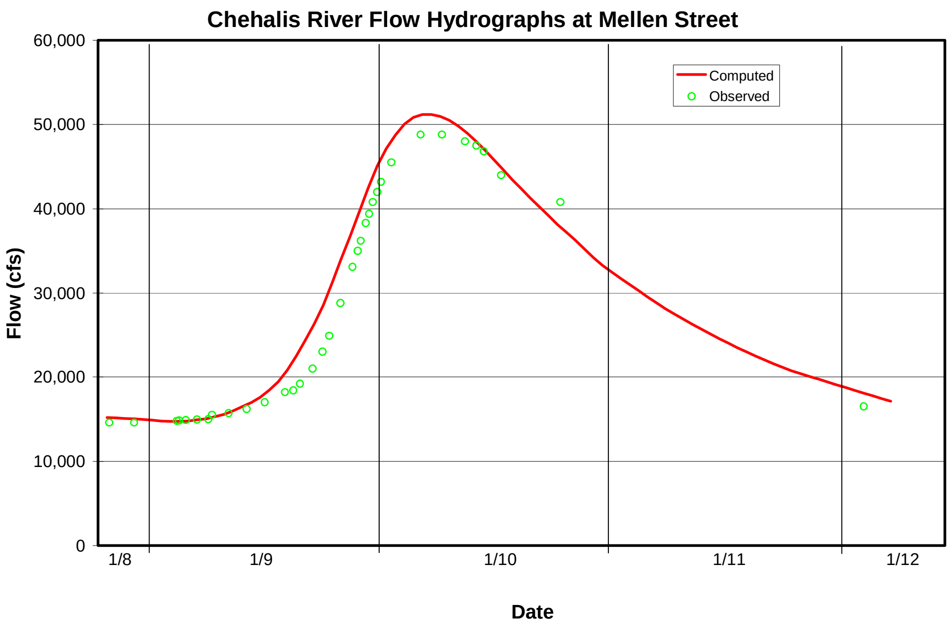


Figure 3-16: Comparison of Computed and Observed Hydrographs on the Chehalis River at Mellen Street – January 1990 Flood



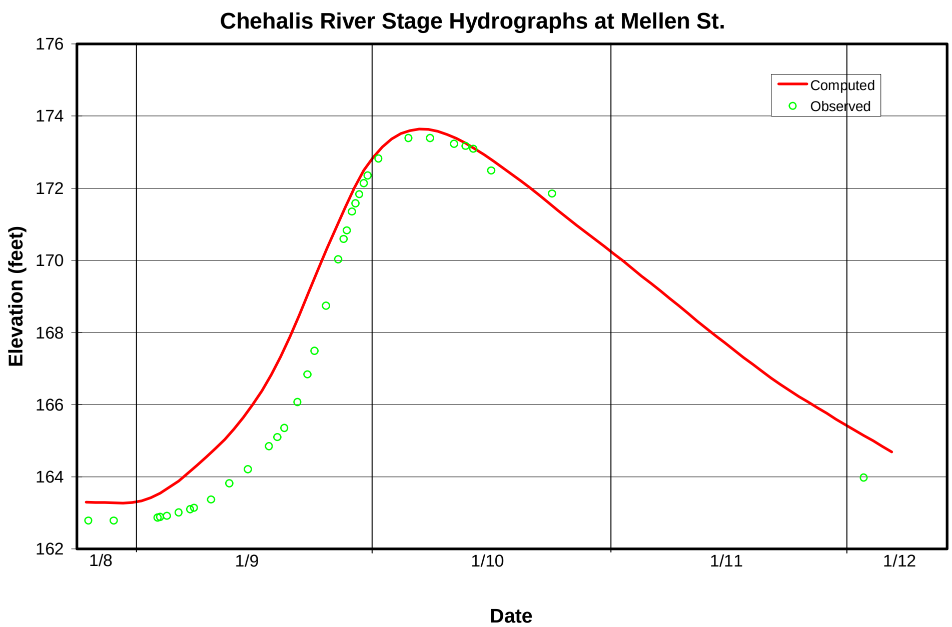
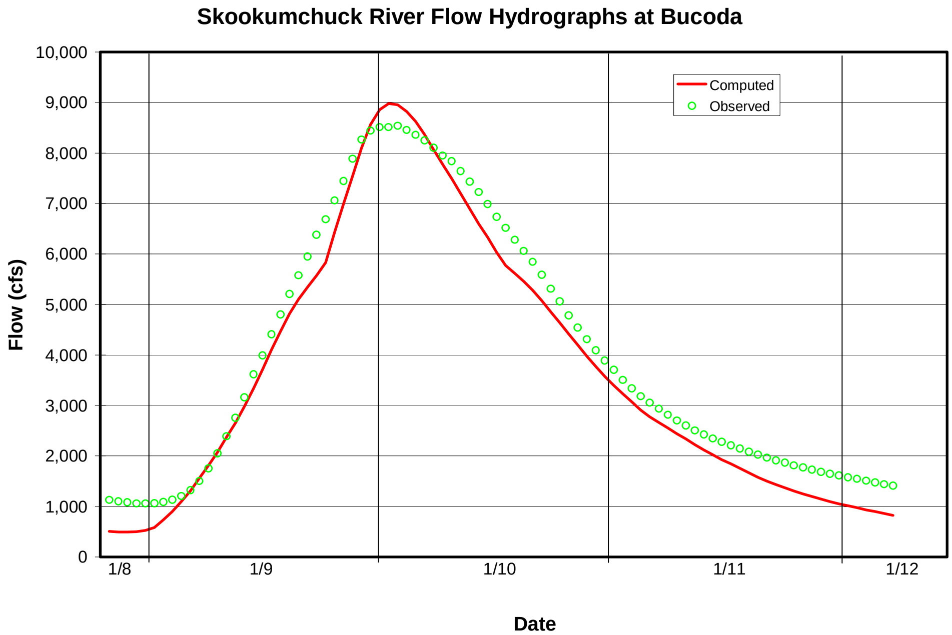


Figure 3-17: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Bucoda – January 1990 Flood



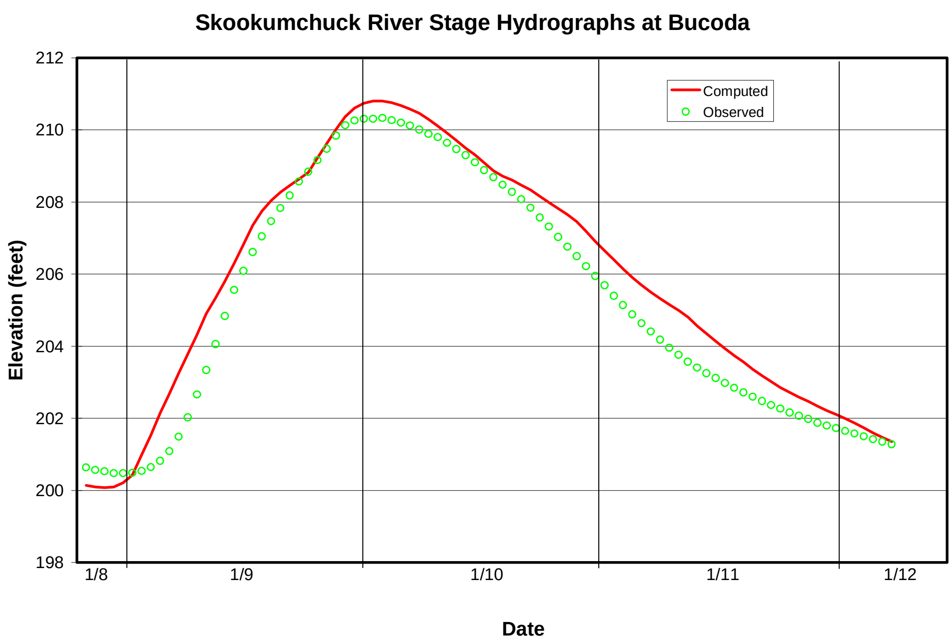
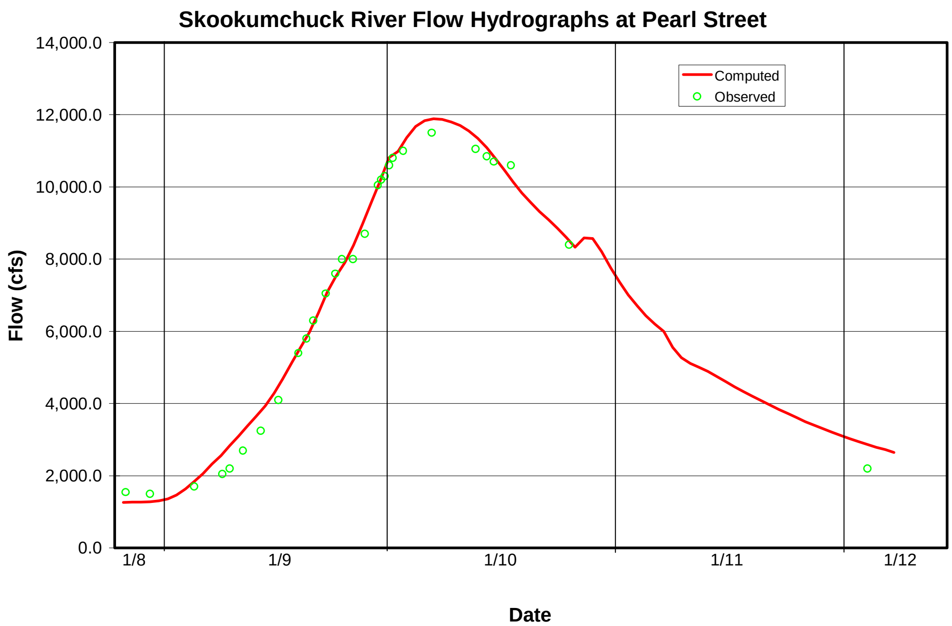


Figure 3-18: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Pearl Street – January 1990 Flood



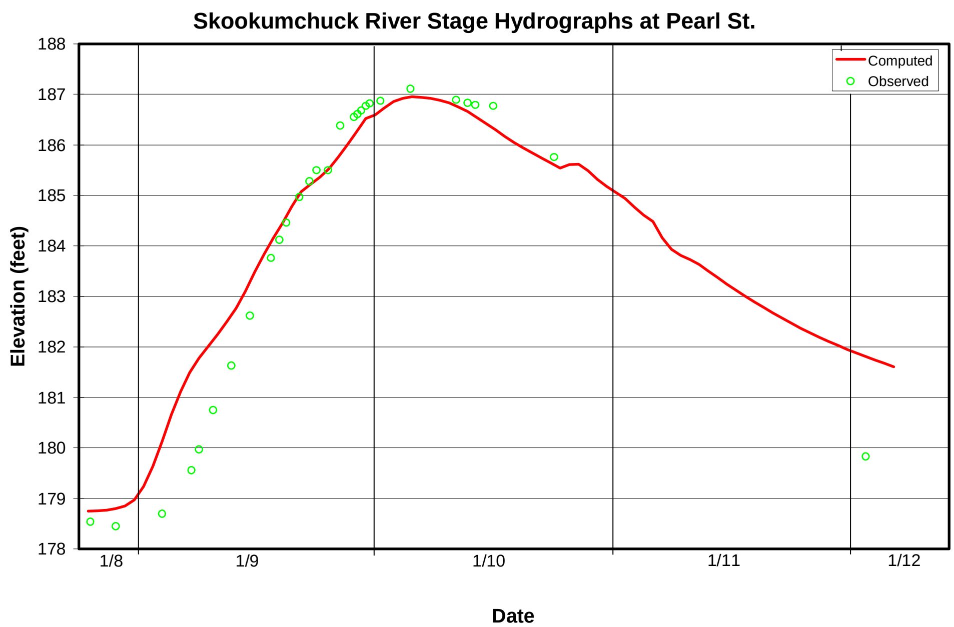
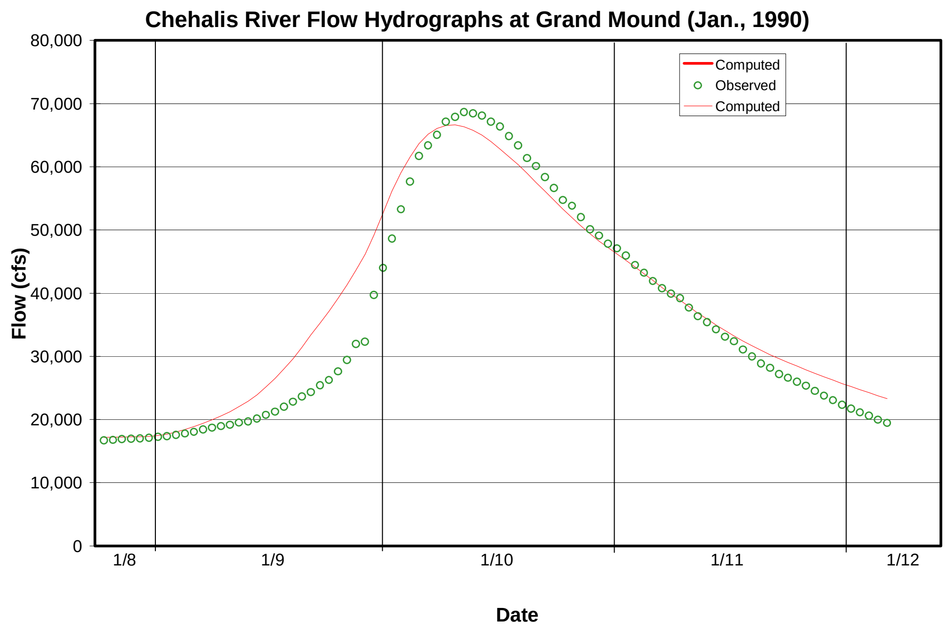


Figure 3-19: Comparison of Computed and Observed Hydrographs on the Chehalis River at Grand Mound – January 1990 Flood



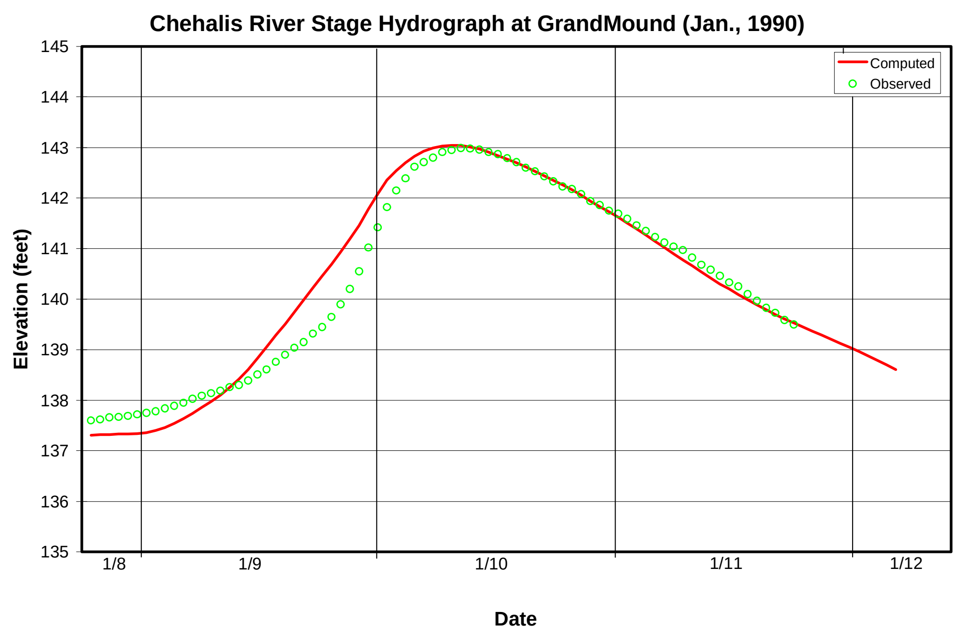
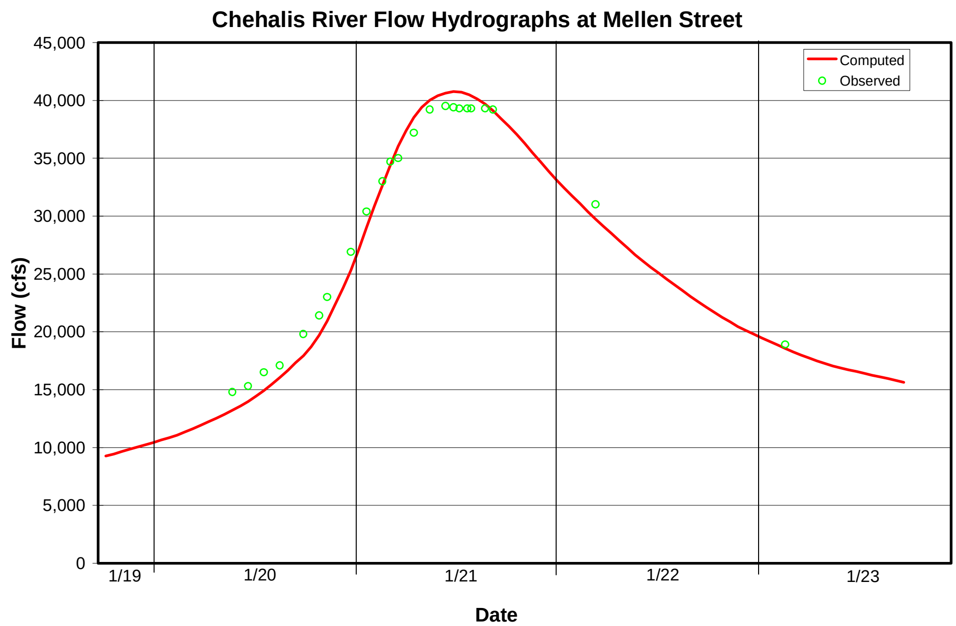


Figure 3-20: Comparison of Computed and Observed Hydrographs on the Chehalis River at Mellen Street – January 1972 Flood



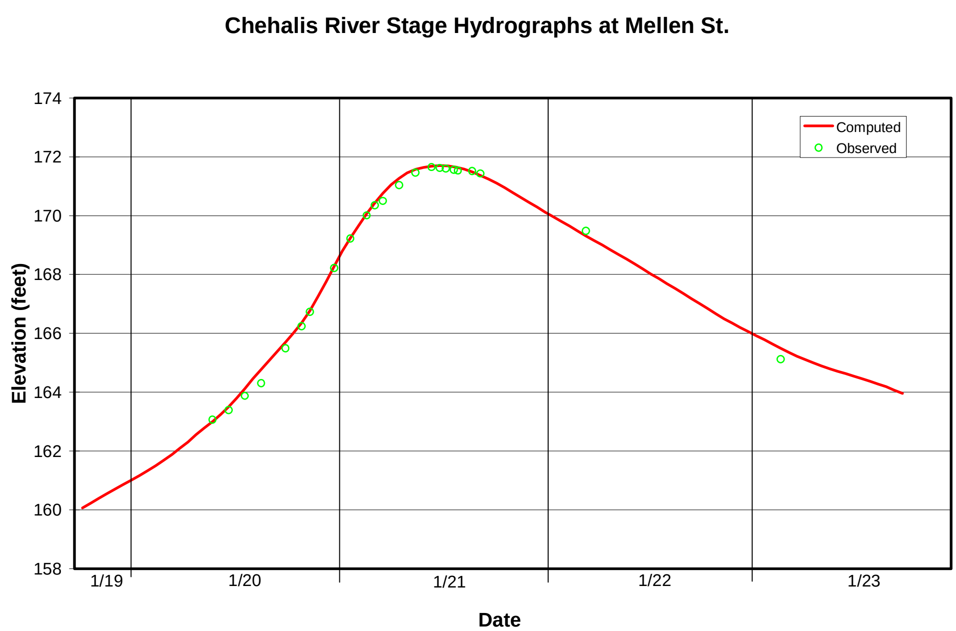
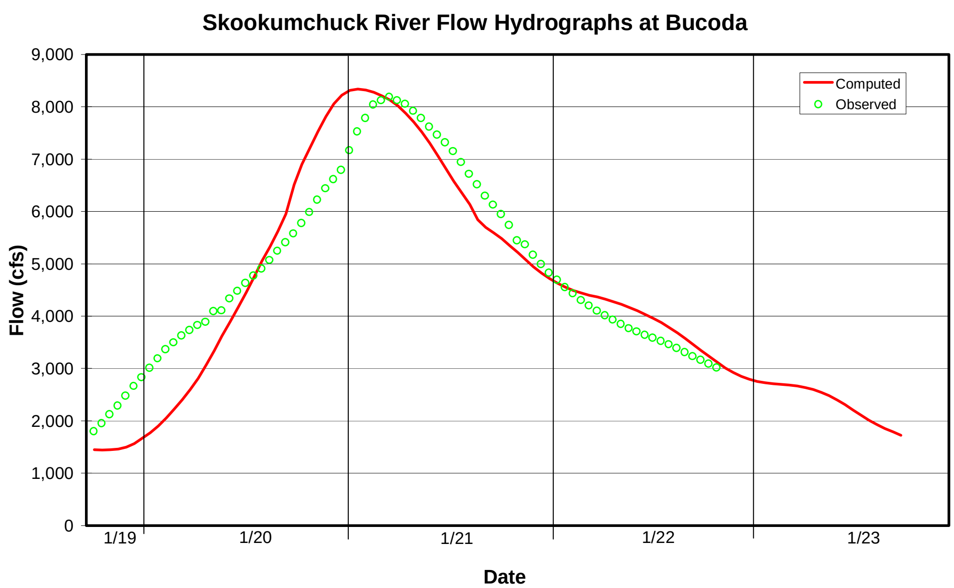


Figure 3-21: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Bucoda – January 1972 Flood



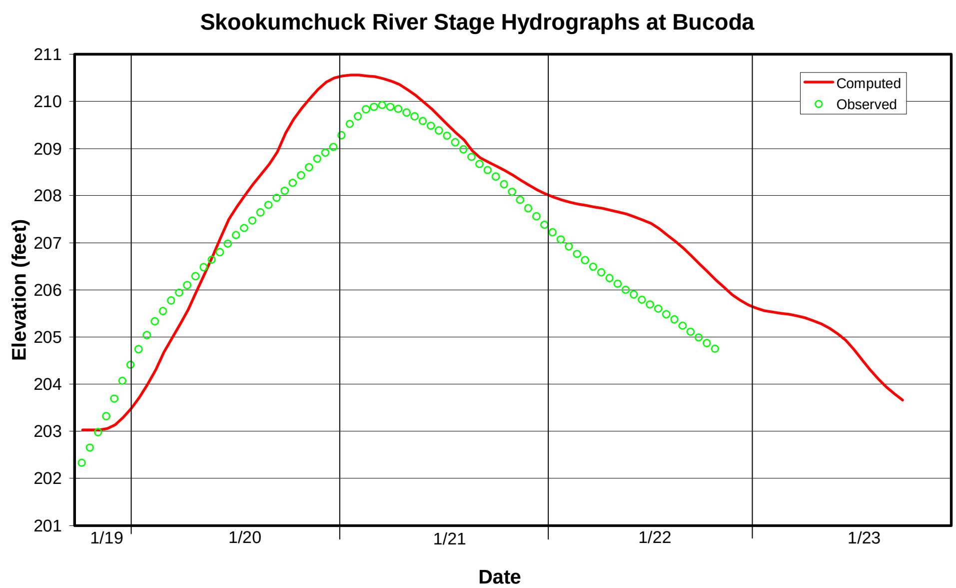
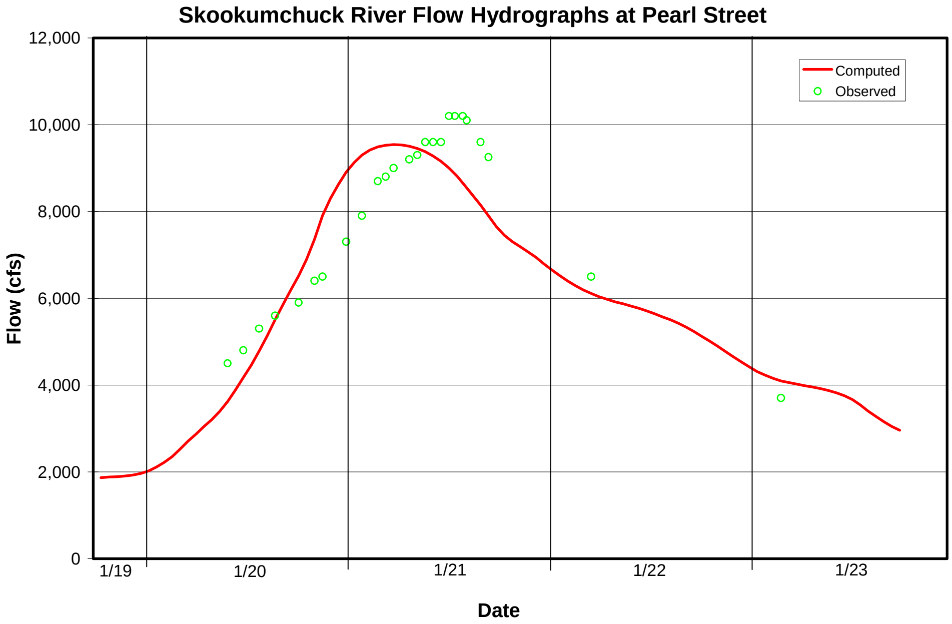


Figure 3-22: Comparison of Computed and Observed Hydrographs on the Skookumchuck River at Pearl Street – January 1972 Flood



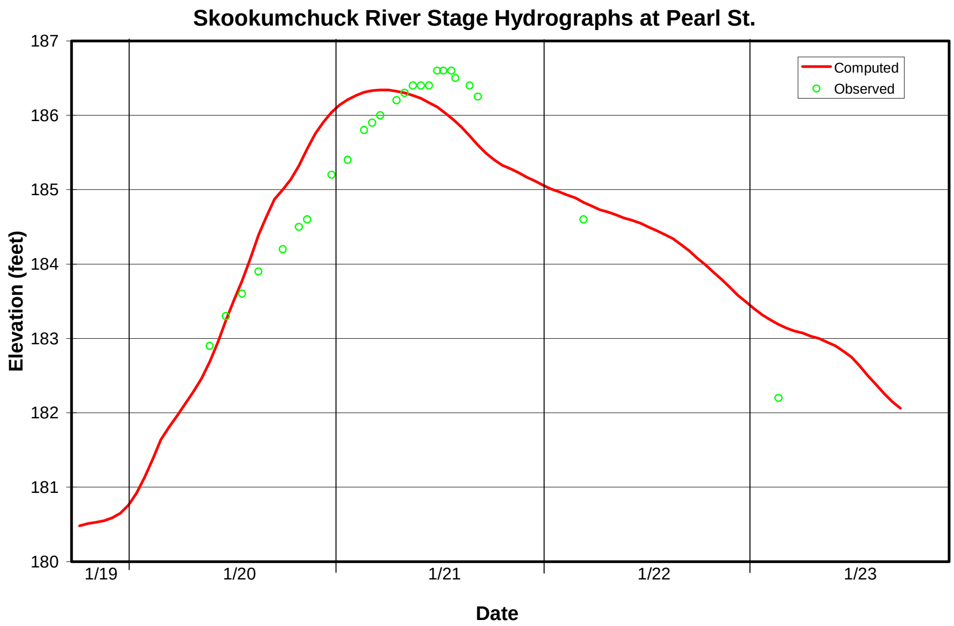
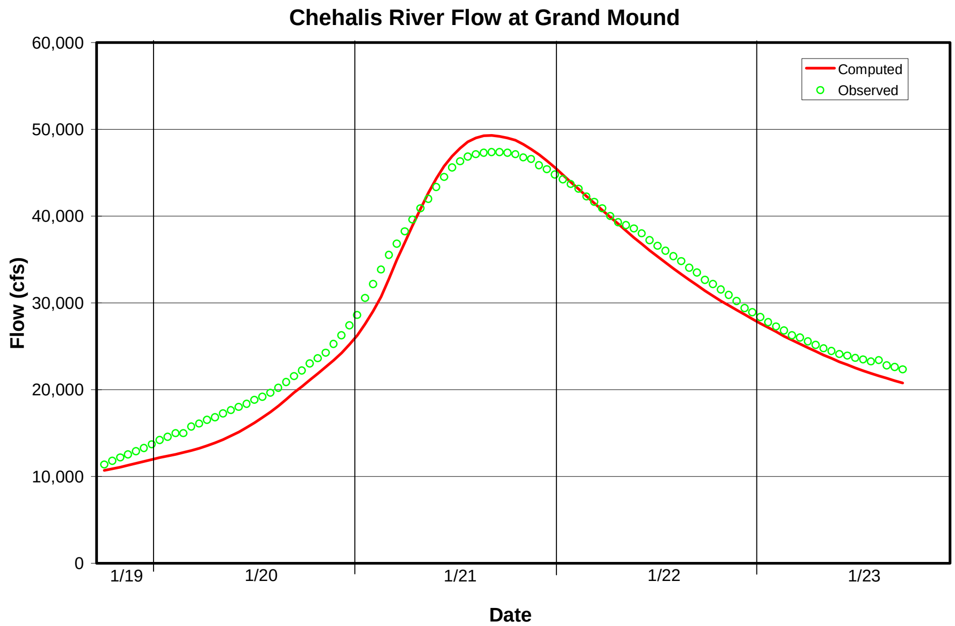


Figure 3-23: Comparison of Computed and Observed Hydrographs on the Chehalis River at Grand Mound – January 1972 Flood



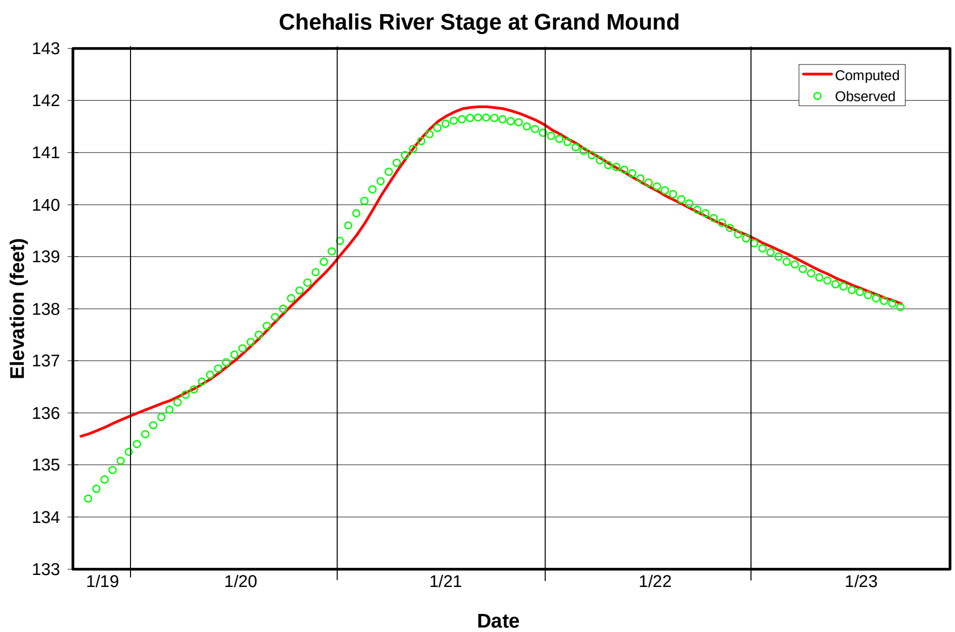


Figure 3-24: Comparison of Computed and Existing USGS Discharge Rating Curve for the Chehalis River at Grand Mound – USACE Statistical Flood Hydrographs

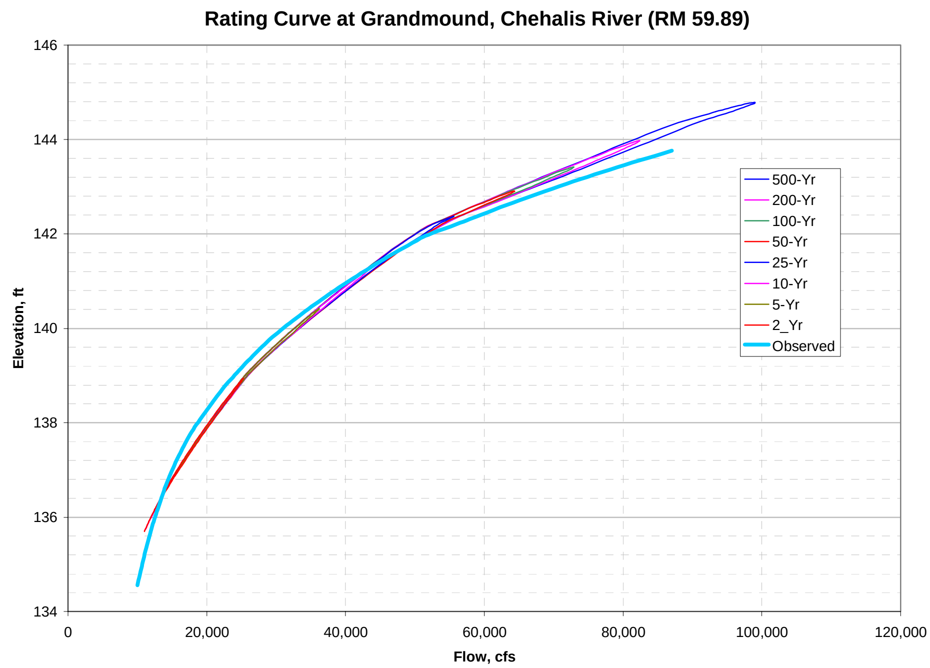
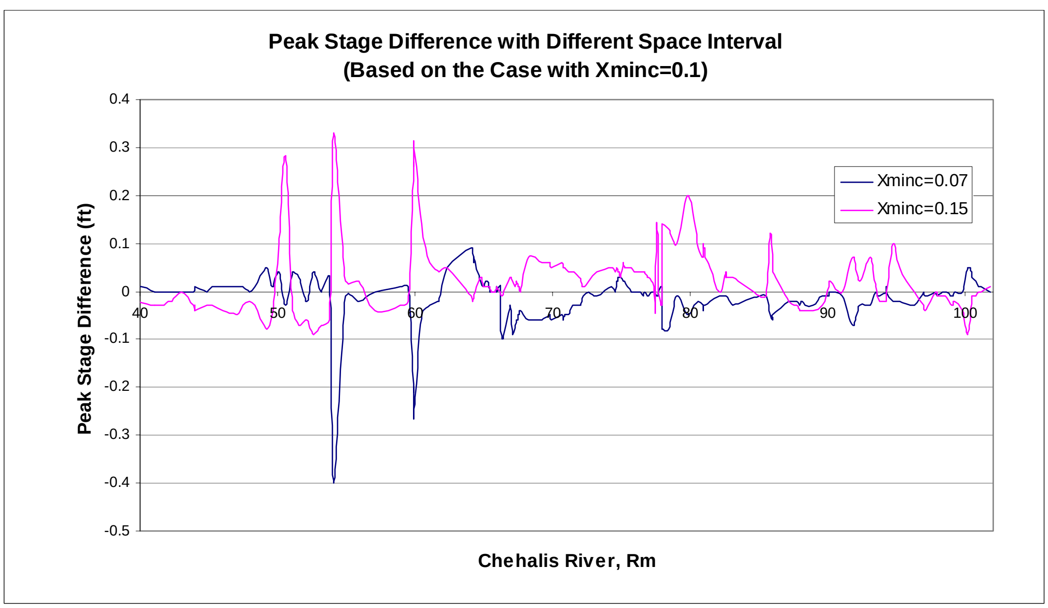


Figure 3-25: Comparison of Distance Step Value - Chehalis River



4. HYDRAULIC DESIGN

4.1 DESIGN CONSIDERATIONS AND CRITERIA

4.1.1 General

Hydraulic design was performed using the general design objectives and criteria for the project as discussed in Section 4.3.4.4 of the General Reevaluation Report. Additionally, a general objective of the proposed levee alignment was that it should reliably protect against the 100-year flood of the Chehalis River. This includes protection from induced backwater flooding from the Chehalis River on tributaries including Dillenbaugh Creek, Salzer Creek, China Creek, Coal Creek, and the Skookumchuck River.

4.1.2 Water Surface Profiles

Water surface profiles are provided in Appendix C, Levee Plan and Civil Design.

4.1.3 Levee Height Analysis

The proposed levees were designed to reliably protect against the 100-year flood. The project formulation adhered to the policy and guidance set forth in ER 1105-2-101, Risk Based Analysis (RBA) for Evaluation of Hydrology /Hydraulics, Geotechnical Stability, and Economics in Flood Damage Reduction Studies. Details of the RBA and formulation can be found in Section 3.4 and Section 4.0 of the GRR. Optimization of the levee height on the Chehalis and Skookumchuck rivers was preformed.

4.2 THE LEVEE PLAN

The levee plan consists of several components; the plan includes a levee system and modifications to the existing Skookumchuck dam. The proposed levee system includes levees along the Chehalis River and its tributaries Dillenbaugh Creek, Salzer Creek, and the Skookumchuck River. In addition to proposed modifications to Skookumchuck for flood control storage.

4.2.1 Levee / Floodwall System

Design of the levee/floodwall system took advantage of opportunities to maximize levee setbacks, allowing floodplain and channel connectivity for environmental purposes. And also took advantage of using floodwalls where traffic control barriers could serve multi-purposes or where it was necessary to reduce the project footprint. The setback levee alignment would protect existing residential and commercial structures, highway and other transportation infrastructure from flooding while not encouraging new floodplain development. Proposed protection would extend along the Chehalis River from approximately RM 75 to RM 64, as well as along most of the lower two miles of both Dillenbaugh Creek and Salzer Creek. In addition, levee protection will be provided on the Skookumchuck River for backwater effects of the Chehalis River. The effected reach is approximately 2 miles upstream on the Skookumchuck to the confluence with Coffee Creek.

The levee system is intended to provide flood protection against the base 100-year Chehalis River flood level with a degree of certainty. The 100-year protection is coordinated with FEMA flood maps, so that they will be compatible. This protection also extends to the tributaries of the Chehalis River. The Chehalis backwater flooding is prevented from going upstream on the following tributaries: Dillenbaugh Creek, Salzer Creek, China Creek, Coal Creek and the Skookumchuck River.

4.2.2 Levee Design Criteria

The standard Corps levee design consists of a 12-foot top width and 2:1 side slopes (2 horizontal to 1 vertical). The fill material must meet the gradation specification and be compacted to Corps standards for levees discussed in paragraph 2.1.3.2. A 6-inch layer of gravel will be placed on the top surface to provide access during flood events and maintenance. Both sides of the levee will be hydro seeded with grass with 4 inches of topsoil over compacted embankment material. Most levees are set back levees, which will not require rock bank protection. For those few areas that do require bank protection. The protection will include 30 inch minus riprap about 3 feet thick, with a 1-foot layer of quarry spalls between the riprap and compacted embankment material.

The concrete floodwall design has a spread footing buried below existing grade; only the vertical portion of the floodwall will be visible after construction. The base width will vary with the height to a maximum of approximately 20 feet and a top width of approximately 1 foot. They will often serve as traffic barriers along the road right-of-way.

As a general rule if the levee or Berm along the highway was less than 1.5 feet a floodwall was used instead of the standard earthen levee.

4.3 Skookumchuck Dam Description

4.3.1 General

Skookumchuck Dam is located on the Skookumchuck River about 12 miles northeast of Centralia, Washington, at Skookumchuck RM 21.9. The dam was constructed in 1970 to supply cooling water to the coal-fired Centralia steam electric plant. The dam has a rolled earthfill central core, buttressed by an earth and rockfill shell. The structure is approximately 190 feet high, with the top of the dam at elevation 497 feet. All elevations referred to in this report are based on NGVD 29 with the 1947 adjustment.

The dam has a 130-foot wide uncontrolled side-channel spillway in a rock cut on the left abutment. The spillway is a concrete ogee with a crest at elevation 477 feet. The spillway invert is at elevation 465 feet. Water passes over the ogee and spills into a 130-foot long by 40-foot wide concrete lined trough. Water then spills down a concrete lined chute. The chute is almost 600 feet long and has a bottom slope that varies from 17 percent to 25 percent. The spillway chute ends in a stilling basin that directs the discharge into a rock cut leading back to the natural river channel.

Facilities are located adjacent to the stilling basin to trap migrating salmon and steelhead for truck transport over the dam.

During low flow months, water released from storage travels downstream to a diversion pumping station at RM 7.3. From there water is pumped through a 3-mile pipeline to the steam electric plant. Under an agreement between the dam owner and state agencies, additional releases are made from the reservoir to supplement flows in the Skookumchuck River to improve fishery habitat.

Outflow from the reservoir is currently either over the spillway crest at elevation 477 feet, or through the outlet works. The existing outlet works consist of an inclined, multilevel intake structure that connects to the construction diversion tunnel and discharges through two 24-inch Howell-Bunger valves into the spillway stilling basin. The intake gates are set at elevations 449, 420, and 378 feet. The discharge capacity of the outlet works is approximately 220 cubic-feet-per-second (cfs) when the pool elevation is at the spillway crest.

Storage behind the dam is essentially a fill and spill operation. The limited outlet capacity of the dam causes the reservoir to fill to the spillway crest at elevation 477 feet early in the flood runoff season. Once the reservoir is full, flood inflow to the reservoir passes over the un-gated spillway, which was originally designed for a discharge capacity of 28,000 cfs with the reservoir pool at elevation 492 feet.

Storage capacity of the reservoir between the normal minimum pool at elevation 400 feet and the spillway crest at elevation 477 feet is 38,700 ac-ft. Additional usable storage of 3,170 ac-ft is available between elevations 378 feet and 400 feet. Dead storage is approximately 1,420 ac-ft between elevations 378 and the base of the dam.

Additional studies that would need to be performed in the next phase of studies would include the following:

- Finalize PMF analysis.
- Detailed numerical analysis of the spillway, chute, and outlet works.
- Structural design of outlet works and spillway and chute modifications.
- Development of flood control regulation rule curves.
- Evaluation of any downstream environmental effects related to reservoir operation and flood control regulation.
- Evaluation of reservoir sedimentation and bank stability.
- Assessment of potential downstream scour and bank erosion.
- Determination of freeboard requirements.
- Assessment of downstream fish passage.
- Evaluation of cavitation potential.

4.3.2 Proposed Dam Modifications

The proposed alternative would consist of constructing a short outlet works tunnel in the left abutment of the dam between the existing spillway and dam crest. An outlets works tower with slide gates would be built at the entrance to the new tunnel. The tunnel would discharge into the existing spillway chute. For the high flood storage pool option, three steel tainter gates would be added to the top of the existing ogee spillway. See Appendix B, Skookumchuck Dam Modifications, for conceptual drawings of the alternative.

Feasibility level hydraulic analyses have been conducted for the outlet works and spillway to determine the approximate configuration and dimensions of the project components necessary to fulfill the project design objective.

The intake structure would be constructed just upstream of the right abutment of the existing spillway bridge. The intake would lead to a short tunnel constructed in the rock forming the left abutment of the embankment dam. The intake would have two control gates and two guard gates. The slide gates would be approximately 8 feet by 11 feet in size. A 3,000 cfs capacity at minimum reservoir pool was used for the preliminary sizing of the gates. Capacity of the outlet works would be approximately 8,000 cfs at the maximum reservoir pool.

Flow would discharge through the tunnel into the existing spillway chute. The outlet tunnel and spillway chute confluence will be a very complex feature to hydraulically design and analyze and a physical model investigation may be required in the final design phase.

The existing uncontrolled overflow spillway would also be modified for the proposed alternative. A few different options were considered for providing spillway crest control including an inflatable rubber weir and steel bascule gate. For purposes of costs and preliminary engineering, it was decided to go with steel tainter gates. There would be three steel tainter gates approximately 39.3 feet wide and 15 feet tall on the spillway crest with two concrete piers between. The spillway would have a total capacity of approximately 25,500 cfs at maximum reservoir pool. The total capacity of the outlet works and spillway would be approximately 32,500 cfs at maximum reservoir pool.

4.4 Reservoir Regulation Considerations

4.4.1 Existing Operations

Skookumchuck Dam currently operates on a fill and spill regime. The reservoir fills each year with the first heavy rains of the fall and then allows all subsequent inflow to spill uncontrolled over the dam until summer when the reservoir lowers as inflow drops.

The existing flow management agreement between PacifiCorp and the Washington Department of Fish and Wildlife (WDFW) for Skookumchuck Dam was completed in May 1998 and is intended to provide benefits to downstream fish resources and the needs of the Steam Plant. There are also provisions for steelhead production and other requirements unrelated to water control. The agreement specifies minimum flows throughout the year, water temperature objectives, reservoir elevations, as well as water use limitations and general guidelines for ramping, coordination and operations. There is no existing flood control capacity at the dam. In the summer, inflow drops off and causes the reservoir to lower until such time the fall or winter rains arrive and fill the reservoir.

Water discharge from the existing outlet tunnel is dependant on reservoir elevation. As the reservoir rises and reaches each intake, the corresponding outflows adjust on a continuum from 95 cfs with one outlet submerged, 140 cfs with two outlets submerged and as much as 220 cfs with all three outlets submerged. After the reservoir fills, discharge is passed both through the sluiceways and over the spillway. Although it varies each year, monthly outflow averages generally range between 95 cfs and 1200 cfs depending on the month. During high flow conditions, discharge from the dam can greatly exceed monthly averages with a 5-year event passing 4,000 cfs and a 100-year event passing 7,425 cfs.

4.4.2 Flood Control Operations

Modifications to Skookumchuck Dam are intended to support limited flood control operations at Skookumchuck Dam. Specifically, reservoir operations will change to allow drawdown in the fall to elevation 444 by early November. It is anticipated that this flood control capacity will remain until a flood event occurs. During a flood event, outflows from the dam will be reduced in order to prevent flow at Pearl Street from exceeding 5,000 cfs. Depending on the magnitude of the event, discharge will be limited to no more than 3,000 cfs. After the event passes, water stored in the reservoir will be released at volumes high enough to reach but not exceed 5,000 cfs at the Pearl Street river gage in Centralia.

Discharge from the project would be via two new 8-foot by 11-foot slide gates located on the dam with a bottom elevation of 436 and a common discharge tunnel entering into the existing spillway on the right bank. The gates purpose will be to pass flood flows through the flood season. The maximum storage pool elevation will be 492 and would require the use of spillway crest control gates. The spillway control gates would be utilized only during events that would require use of the additional flood control storage. This additional storage would be reserved for flood above the 70-year event and not fully utilized until around the 100-year event.

4.4.3 Routine Operations (Post Construction)

In the absence of a flood, Skookumchuck Dam is expected to operate for the benefit of both PacifiCorp, and the natural resources of the River. However in the existing operations guidance, not all areas of routine operation are clearly described. For instance, there is little discussion of proper ramping rates. The WDFW/PacifiCorp agreement of May 1998 simply states: “Flow reductions under this Agreement shall be accomplished in a manner that minimizes the stranding of juvenile fish”. Specific criteria were not provided initially because the bypass reach between the dam and its hydropower unit was so short and no other opportunities to significantly modify flows existed at the dam. With the installation of flood control capability however, large changes in river stage will become possible.

Other Western Washington flood control projects were reviewed on order to develop more specific guidelines. This review revealed that both up and down ramping should contain restrictions based on the season and fish resources. With the exception of special operational needs, routine ramping rates between projects were reasonably consistent

In addition to the ramping rates for routine operations, several specific criteria were described for times of flood control or sensitive spawning periods. For instance, ramping rate guidelines for Mud Mountain Dam are more flexible during times of flood control where the tailwater elevation may increase as much as 1 foot/hour. It is however, specifically requested that great consideration be given to public safety prior to changes of that magnitude. At Howard Hanson Dam, special ramping criteria are given during the steelhead spawning and incubation periods (April- July). To protect eggs incubating in redds near the river margins, ramping is not allowed to alter river stage greater than 1 foot below the highest average mean daily flow for the previous 10 days.

4.4.4 Reservoir Operations

Post project reservoir operations will be tied primarily to flood control where a requirement will be in place to ensure the reservoir elevation is at or below 444 prior to the onset of flood control season in early December. During the summer to fall drawdown period, flows from the project will be passed through the outlet structures such that the reservoir lowers to elevation 444. When drawdown is complete, inflow will be passed through the outlet works to maintain reservoir elevation so long as flows at Pearl Street remain under 5,000 cfs. It is expected that project discharges would meet or exceed the minimum instream flows of 90 cfs except if reservoir inflow fell below 90 cfs. The reservoir should remain relatively constant throughout the late spring, summer and early fall. In winter, larger reservoir fluctuations may occur as the project reacts to flood events and the reservoir fluctuates between elevations 492 and 444.

4.4.5 Downstream Flows

Flow operations from Skookumchuck Dam during non-flood events will be similar to the operation that is in place today. Except for flood events, post-project outflows should continue to follow historic outflows as recorded by the Bloody Run gage located slightly downstream of the dam.

The Bloody Run gage shows wide flow variations through the years. In general, daily discharge trends show flow increasing from a low of about 100 cfs in the late summer (August) to a mean monthly flow in January and February around or exceeding 1000 cfs. This pattern can vary widely by year although the summer month regimes are quite consistent.

Maximum flows can be much higher than the average mean of around 1000 cfs. During flood season, high water releases of between 2000 and 3500 cfs are not uncommon. These events tend to be relatively short in duration lasting around 4 to 6 days. Bankfull flows in the upper reaches but below the dam occur at discharges of 3,000 cfs.

4.5 Recommendations for an Operations Plan

Beyond describing and identifying potential biological benefits and impacts of providing flood control at Skookumchuck Dam, is it the goal of this report to propose a plan for operating Skookumchuck Dam. The operating plan developed here is designed to take into consideration the environmental conditions at the site and provide for their protection. The recommendations below are proposed for consideration and review in hopes that they provide a basis for operating Skookumchuck Dam for the highest practical protection of biological resources.

4.5.1 Flood Control Rule Curve and Discharges

The development and adoption of a rule curve is a major operational feature associated with the addition of flood control at Skookumchuck Dam. The rule curve guides decisions on dam releases during flood control operations as well as guiding the rate of reservoir evacuation.

The rule curve also serves as a guide for refill and drawdown planning. Since a rule curve affects reservoir elevation and downstream releases so significantly, it should be developed with consideration for biological resources.

Initial discussions with hydrologists at the Corps resulted in the development of a provisional rule curve based on initial review of flood control data and the biological information provided in earlier sections (Figure 4-1). While it is not a formal and binding rule curve, it does provide a proposal for the protection of biological resources. The provisional rule curve was based on the following assumptions

- Flood storage drawdown to provide at least 11,000 ac-ft.
- Refill initiated based on water forecasts but completed by April 1
- Drawdown initiated when inflow to reservoir is less than instream minimums or when necessary to ensure drawdown by target date of October 31.
- Minimum instream flows are 95 cfs (Nov 1 - Sept 9) and 140 cfs (Sept 10 - Oct 31).
- Minimum pool is 455.
- Maximum pool is 477.

Based on the information available at the time of this report, it is recommended that the provisional rule curve be used as a starting point for hydraulic evaluation. Although it is recognized that the final rule curve may deviate from this provisional rule curve, the curve is considered to be consistent with the most significant biological needs of the system and where changes are made, the rational for the deviation should be documented.

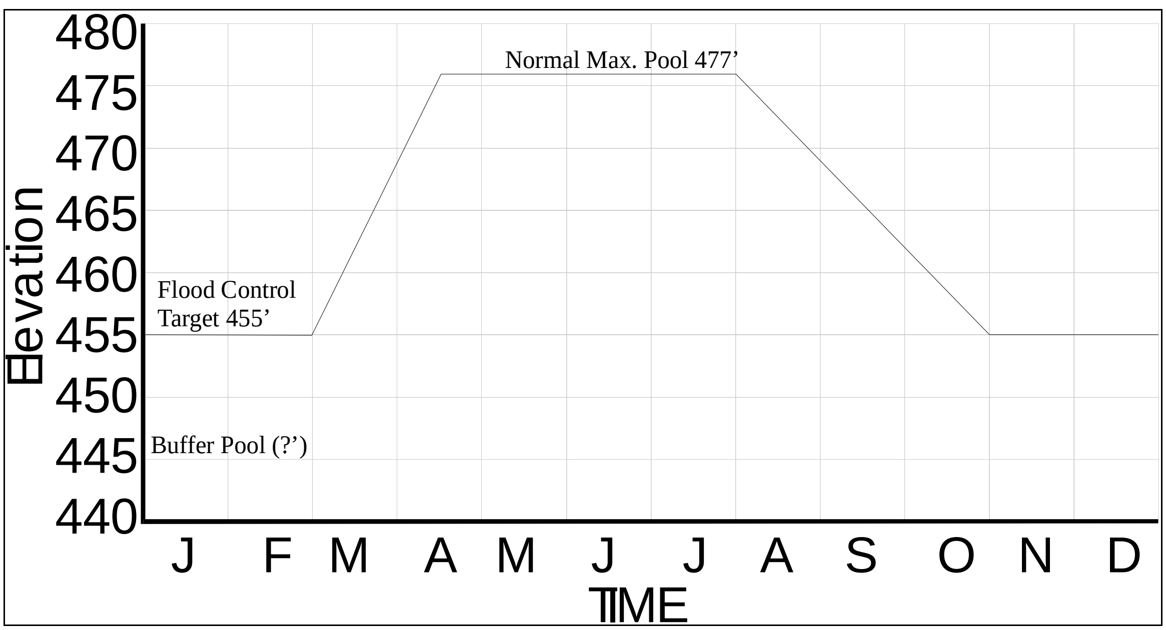


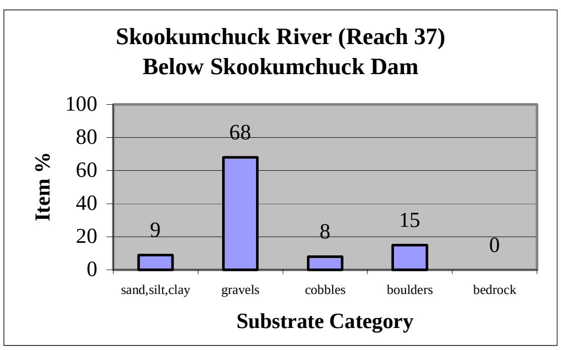
Figure 4-1: Provisional Rule Curve for Skookumchuck Dam

4.5.2 Maximum flows

Rule curves are paired with flood control objectives and forecasted inflows to regulate discharge from the dam for the purpose of managing high flow events. For the Skookumchuck River, discharges above the 2-year event are captured within the reservoir and held to ensure the Pearl Street objective of 5,000 cfs is not violated. After the peak of the high flow event passes, stored water is released to remain within the Pearl Street objectives until the reservoir is evacuated and ready for the next event. The evacuation of the reservoir adds additional flow to the end of each event extending bank full flows beyond the baseline condition. The impacts of this are described in earlier sections but it appears there are two significant physical considerations when managing these high flows. First it is critical that the existing gravels and fines be allowed to continue moving towards the Chehalis River. Bedload movement and channel scour processes are critical to maintaining spawning gravels, woody debris recruitment, undercut banks and other mainstem habitats. Secondly, it is critical to ensure the reduction in high flows to levels at or below the 2-year event will allow for adequate maintenance of important off-channel habitats.

4.5.3 Bedload Movement and Channel Processes

Bedload characteristics of the Skookumchuck River are predominantly gravel and cobble. The results of pebble counts done in 2000, showed no clear trend except that larger substrate types were found closer to the dam and finer materials tended to show up down towards the mouth or in flat reaches such as near the town of Bucoda (Figures 4-2 and 4-3)



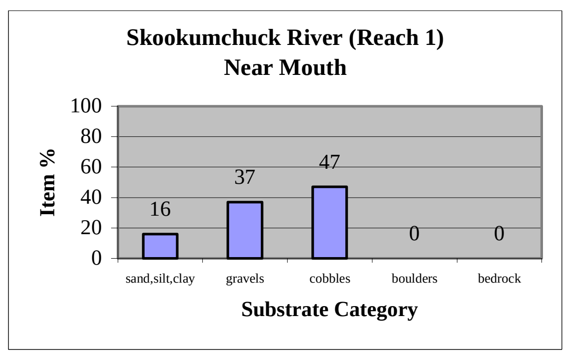


Figure 4-2: Pebble Count Data at Reach 37 (left)
Figure 4-3: Pebble count data at Reach 1 (right)

The disposition of substrate after the proposed flood control operations are in effect will be linked to the rivers ability to recruit and move material downstream. The flood control project proposes limiting downstream discharge to the 2-year flood, which represents a restriction to higher flows from the existing condition. The pebble count data from the Skookumchuck River appears to indicate that Skookumchuck Dam may be restricting some gravel recruitment from the upper watershed but that the input of gravels from tributaries such as Bloody Run Creek, Hanaford Creek and others are currently providing gravel adequate for spawning by anadromous fish. The size and distribution of the gravel appears to be small enough to allow mobilization and transport at moderate to small flood flows such as the 2-year event although more information may be needed to confirm this. Although it is less clear whether the 2-year event will allow for enough movement often to prevent gravel build up at tributaries mouths. A common problem in small to medium sized tributary systems, the Skookumchuck appears large enough to make the 2-year flood flow potential (3,000 to 5,000 cfs) likely to move the gravel size common within its banks and manage the deposition potential at the mouths of the tributaries.

Similarly, the 2-year event is expected to continue the processes of erosion on unstable and unconsolidated banks. Also, the reduction in peak flows may lengthen residency times for woody debris which will likely offset initial limitations to woody debris recruitment from reduced flood discharges.

Based on the above information, it is recommended that the proposed project allow the 2-year event be passed and not stored for flood control or other purposes. The 2-year flood events appear vital to the maintenance of the Skookumchuck River channel and may be particularly necessary in the Bucoda reaches. It is further recommended that no levee structures be constructed that limit the ability of the 2-year event to deposit and erode channel materials.

4.5.4 Minimum Flows

Minimum flows are dictated by the ability of inflow to support the existing summer requirement of 95 cfs. In most years, inflow appears capable of meeting or exceeding this standard. When minimum flows are elevated to 140 cfs between September 1 and October 31, inflows are not always able to meet the demand. The impact of this lies principally to adult chinook salmon which migrate upstream during this time. Inadequate flows during this period may increase travel time and availability of spawning habitat. The WDFW has informally expressed an interest to improve flows between September and October from 140 cfs to 160 cfs to ensure adequate flows for adult chinook. The difficulty rests in getting additional water without impacting resources during other times of the year. The provisional rule curve provides for drawdown to October 31 thereby allowing flexibility to provide some additional water during this period depending on water availability and reservoir management. Additional efforts are needed to provide insight into the reliability of providing additional water in the late summer.

An engineering study should be conducted to investigate the possibility of storing water to allow an additional release of 20 cfs to increase the minimum flows between the months of September 1 and October 31. It is recommended that this study include discussion of impacts to flows elsewhere during the year and the reliability of providing the water. It is also recommended that the existing minimum flows criteria of 95 cfs be maintained and not reduced in support of this action.

4.5.5 Ramping Rates

An expansion on the limited guidance given in the PacifiCorp and WDFW agreement appears to be warranted. The operation plan recommends using guidance from elsewhere to ensure river levels are manipulated such that they minimize concerns over fish stranding or spawning impacts but no specifics are given.

In reviewing projects with established ramping rate criteria, it appeared the ramping rates reflect a high degree of consistency and giving some confidence that ramping rates could be transferred between projects and remain adequate for resource protection (Table 4-1). There are however, discrepancies within our examples. These were seen in the areas of winter daytime ramping rates as well as spring ramp up rates (both daytime and nighttime). Also, the seasonal calendar is different between the two projects with the dates June 1- June 16 included in the spring period for the White River.

For the calendar discrepancies, it is recommended that the early June period remain within the spring ramping period to ensure ramping rates are sufficient for late outmigrating steelhead. Similarly, it is recommended that conservative daytime spring ramp-up rates be adopted for the protection of juvenile protection. It is also recommended that a 2-inch/hour ramping ability in the nighttime be allowed for quicker maintenance operations and minimal disruption to steelhead spawning.

Table 4-1: Recommended Ramping Rates for the Skookumchuck River

Season	Direction	Time	Rate
February 15-June 15	Up	Day	0”/hr
		Night	2”/hr
	Down	Day	No ramping
		Night	2”/hr
June 16-October 31	Up	Day	1”/hr
		Night	1”/hr
	Down	Day	1”/hr
		Night	1”/hr
November 1-February 14	Up	Day	1”/hr
		Night	1”/hr
	Down	Day	2”/hr
		Night	2”/hr

4.5.6 Upstream Fish Passage Operations

Upstream fish passage operations are limited to the passage of adult steelhead around the dam between the months of September 15 and November 15. The option to haul coho and chinook remains open but the current focus is to rebuild steelhead populations before allowing additional salmon above the dam. In addition, most spawning habitat for chinook was lost through the creation of the reservoir. The operation is conducted at a fish trap located at the base of Skookumchuck Dam. The trap appears to be adequate for its purposes. The need to provide access to the productive habitats of the upper Skookumchuck watershed is recognized and it is proposed that the operation continue with one modification.

To ensure the adult steelhead continue their upstream travels with a minimum of disruption, it is recommended that they be transported and released above the upper end of the reservoir. The release site should also be maintained to minimize injury and fallback. It is also recommended that they transfer truck be maintained in good condition with proper aeration equipment.

4.5.7 Downstream Fish Passage Operations.

Downstream passage occurs primarily between the months of April 15 and May 31 with juvenile steelhead as the only anadromous outmigrant. To date, there are no other juvenile anadromous salmon above the project. The existing downstream passage plan for Skookumchuck Dam relies heavily on a full pool condition arising prior to the onset of outmigration. Full pool allows outmigrants access to the spillway and the fish passage chute both designed to pass fish via surface flow down the spillway and into the river below. During periods of use, the existing outlet gates are also a potential source of exit for the outmigrants. They are located within a reachable depth but probably don't exhibit enough attraction to induce many fish to use the outlets for passage. A new gate located adjacent to the dam would likely attract more fish than the existing outlet gates.

Based on the need for a full pool to pass juvenile fish most successfully, it is recommended that the pool be refilled at the end of the flood control period or no later than April 1. This condition should be allowed to continue until natural inflows cause the reservoir elevations to drop. It is critical to design any new outlet gates and tunnels such that safe fish passage through that structure can be assured. In years of drought, the reservoir may refill slowly or not at all and increase the use of outlet gates for outmigration. Similarly, the potential for forecasting late flood events may cause the reservoir to remain evacuated, delaying refill and increase the use of outlet gates for outmigration.

4.6 UNET Hydraulic Model

To evaluate the potential effects of various flood control alternatives in reducing flood stages and corresponding damages in the Centralia-Chehalis floodplain, a baseline flood model was developed. The baseline flood model represents the existing conditions of the Upper Chehalis River Basin above the Porter gage including the recent completion of the Long Road Dike Project construction in February 2001. A complete discussion of the UNET hydraulic model developed for the Chehalis River can be found in Section 3.

The calibrated UNET model of the Chehalis River was modified to incorporate the levee alternative elements including levee segments along the Chehalis River, Skookumchuck River, Salzer Creek, and Dillenbaugh Creek, flood control boxes in Dillenbaugh Creek, and modifications to the hydrographs input to Reach 14 of the UNET model to represent the proposed flood control operations at Skookumchuck Dam. Eight flow events were modeled, ranging from the 2-year to the 500-year event.

4.7 Modification of UNET Model

4.7.1 Skookumchuck Dam Modification

Two options for Skookumchuck dam modification were examined for this alternative. Option 1 would provide flood control storage of approximately 11,000 ac-ft between pool elevation 455 and 477 feet. Option 2 would provide flood control storage of approximately 20,000 ac-ft between pool elevation 455 and 492 feet. Future reservoir operations based on these two options were simulated using a reservoir operations model. Output from the reservoir model (time-series of simulated discharge from the reservoir) was used as an input hydrograph to Reach 14 of the UNET model to represent reservoir discharge to the Skookumchuck River. The outflow hydrographs from the dam for the proposed reservoir operation scenarios were developed for eight flood events. For the flood events less than or equal to the 50-year flood, the outflow hydrographs would be the same for both options. For flood events larger than or equal to the 100-

year flood, the outflow hydrographs would be different for the two options (this suggests that flood storage in the reservoir above elevation 477 feet would only be utilized for flood events on the order of a 100-year flood event or larger). A detailed discussion of flood control operation can be found in Technical Report 2.

4.7.2 Levee Segments

Changes made to the UNET model to represent the proposed levee segments included modification of topographic information to represent the levee alignment and elimination of certain hydraulic connections between river reaches and off-channel storage areas. For instance, channel cross-sections depicted in UNET were modified where appropriate to represent the proposed levee system. The proposed levees were designed to reliably provide protection against the base 100-year flood level. Hydraulic connections modeled in UNET between river reaches and off-channel storage areas were eliminated as appropriate to simulate the proposed levee alignment. For instance, hydraulic connections in the model that allow the transfer of water between the Chehalis River and the Chehalis airport area under existing conditions were removed from the model since proposed levee segments around the airport are designed to prevent future flooding in this area.

4.7.3 Flood control boxes

Two flood control boxes were added at RM 0.623 and RM 0.064 of Dillenbaugh Creek. The flood control boxes were operated during the flood to prevent the city of Chehalis from being inundated.

4.8 Hydraulic Modeling Results

4.8.1 Alternative 4 only

Modeling of Alternative 4 indicates that most of the urban flooding in the vicinity of Chehalis and Centralia from the Chehalis River and its main tributaries (i.e., Skookumchuck River, Salzer and Dillenbaugh Creeks) would be eliminated under Alternative 4 during flood events up to at least a 100-year magnitude. Most of this benefit is derived from the proposed system of setback levees, which will protect Interstate 5 as well as much of the urban areas to the east of I-5 from flooding. Reduction of flooding from the Skookumchuck River would be limited to areas along the most downstream reach of the river where levees are proposed as part of this alternative. Peak stages and associated flooding along the Skookumchuck River upstream of the levees would be the same as under existing conditions.

Alternative 4 is expected to have little change to the peak stage within the Chehalis River and its tributaries relative to existing conditions because most of the proposed levees are setback significantly from the affected river channels. For instance, levees proposed along the Chehalis River floodplain will be limited to the right (east) bank of the river and will be setback from the existing banks sufficiently to have little impact to the active portion of the floodplain. As a result, active floodplain areas where most of the flood flow is typically conveyed during flood events will still function as they currently do under existing conditions. The primary function of the proposed levees will be to eliminate flooding of I-5 and primarily urban areas (mostly on the east side of I-5) that have historically functioned as backwater storage areas during flood events but have had very limited function in terms of providing downstream conveyance of flood flows. For instance, UNET modeling of Alternative 4 suggests a slight increase in the peak stage in the Chehalis River downstream of roughly RM 74 during a 100-year flood event, with a maximum increase of about 5 to 6 inches in the vicinity of RM 72 to 73. It should be noted that these potential stage increases are in addition to peak 100-year flood depths on the order of 5 to 10 feet or greater within this reach under existing conditions. Furthermore, these potential stage increases would be limited to floodplain areas that would not be protected by the proposed levees such that only a small number of homes would be affected.

The UNET modeling also suggests the potential for slight increases in peak flood stage in the Chehalis River downstream of the project area as a result of Alternative 4. However, projected increases in the 100-year peak stage are roughly one-tenth of a foot or less, which represents a stage increase that would be virtually undetectable and essentially insignificant when compared with peak stage and flood depths under existing conditions.

Along the lower four miles of the Skookumchuck River (vicinity of Centralia), peak flood stages would decrease in the range of 0.5 to about 1.5 feet relative to existing conditions during a 100-year flood event for the combination of Option 1 Skookumchuck reservoir flood control with Alternative 4. These estimated reductions in peak stage are attributable to proposed improvements to flood control operations at Skookumchuck reservoir.

4.8.2 Alternative 4 and Skookumchuck Dam Modifications

Two options of Skookumchuck Dam operation for flood control purposes were used in combination with Alternative 4. The addition of improved flood control operations at Skookumchuck dam has two primary benefits to Alternative 4. First, improved flood control operations at the dam would provide flood reduction benefits to the Skookumchuck River valley downstream of the dam. Second, while flood control operations at the dam would provide limited flood reduction benefit to the Chehalis River valley, there does appear to be a sufficient reduction in flood stage to offset any potential stage increases attributable to the proposed levee system.

For the combination of Option 1 Skookumchuck reservoir flood control with Alternative 4, the peak flood stage in the Chehalis River would decrease relative to existing conditions over most reaches. For instance, UNET modeling suggests slightly lower peak stages in the Chehalis River downstream of RM 71 relative to existing conditions. The peak flood stage in the Chehalis River would still increase slightly between RM 71 and RM 74 with a maximum increase of about 5 to 6 inches during a 100-year flood. As noted under Section 4.5.1, these potential stage increases are in addition to peak 100-year flood depths on the order of 5 to 10 feet or greater within this reach under existing conditions and would only affect a small number of homes within the floodplain that would not be protected by the proposed levees.

For the combination of Option 2 Skookumchuck reservoir flood control with Alternative 4, the peak flood stage in the Chehalis River would decrease relative to existing conditions over most reaches. UNET modeling suggests slightly lower peak stages in the Chehalis River downstream of RM 71 relative to existing conditions similar to the combination of Option 1 Skookumchuck reservoir flood control with Alternative 4. Similar to Option 1, the peak flood stage in the Chehalis River would still be increased slightly between RM 71 and RM 74 with a maximum increase of about 5 inches near RM 73. As noted above, these potential stage increases are in addition to peak 100-year flood depths on the order of 5 to 10 feet or greater within this reach under existing conditions and would only affect a small number of homes within the floodplain that would not be protected by the proposed levees.

Along the lower four miles of the Skookumchuck River (vicinity of Centralia), peak flood stages would decrease in the range of 0.5 to about 2.0 feet relative to existing conditions during a 100-year flood event for the combination of Option 2 Skookumchuck reservoir flood control with Alternative 4. These estimated reductions in peak stage during a 100-year flood event are slightly greater than the modeled stage reductions attributable to Option 1 Skookumchuck reservoir flood control operations. Estimated stage reductions in the Skookumchuck River in the vicinity of Centralia during flood events smaller than a 100-year event should be the same for either Option 1 or Option 2 flood control at Skookumchuck reservoir due to similar flood control operation.

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